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BUREAU OF PUBLIC ROADS

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STEAM SHOVEL LOADING HAS BEEN FOUND TO BE ONE OF THE CHEAPEST METHODS IN QUARRY OPERATION

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SIMPLIFIED SOIL TESTS FOR SUBGRADES AND THEIR PHYSICAL SIGNIFICANCE

Reported by DR. CHARLES TERZAGHI, Massachusetts Institute of Technology, Research Consultant to the Bureau of Public Roads

ONE OF the outstanding and fundamental problems of subgrade investigation consists in finding methods for identifying soils encountered in a certain locality with other soils obtained from other sections of the country. Suppose an experimental road has been built and a careful investigation has been made of how the road surface behaved under varying conditions of temperature and traffic. All these results are practically worthless unless we are in a position to determine where else in the country identical subgrade conditions exist.

The problem of identifying for engineering purposes the materials of which the soils consist is practically solved, inasmuch as we have already an experimental method which furnishes all the data required. These data concern the compressibility, the swelling, the permeability and the compressive strength of the material. They are in every respect comparable to the data used in structural engineering for expressing the properties of construction materials, and there is no doubt about their physical meaning. The method has recently been developed by the writer, and a set of equipment required for making the tests is being installed at the experimental station of the Bureau of Public Roads at Arlington, Va.

However, since the identification tests last one week for every set of six samples, it is desirable to develop methods for rapid preliminary classification of soils with a view of selecting merely some typical representatives for the more elaborate physical test.

Among the many tests which have been proposed for the preliminary classification of soils, the method devised by Atterberg seems to be by far the most promising one for the following reasons:

(*a*) The tests can be made within a short time, with simple equipment.

(*b*) They furnish three independent coefficients, viz, the liquid limit, the plastic limit, and the shrinkage limit.

However, before we can discuss the relative importance of these tests and the part they are called to play among the methods for soil identification we must find out what these tests mean and what are the factors which determine their results.

The fundamental principle of Atterberg's method is very simple and very logical. Suppose we mix a soil with a considerable quantity of water, thus transforming it into what is called the liquid state. In this state the soil is even apt to flow out like a thick juice whenever it has an opportunity to do so. However, if the water is allowed to evaporate, the consistency of the soil becomes stiffer. It first passes into a state in which it does not flow any more. However, by taking a piece of the soil and working it with the hand, we can readily mold it. We say the soil has become plastic. Further evaporation causes the soil to lose this property and to break whenever we try to mold it. Its state has become semisolid. Finally, its color changes from dark to light and it becomes very hard. (Solid state). Experience shows that the water content at which a soil passes from one of these states into the next one is very different according to the nature of the soil Hence it was plain common sense which suggested the working out of certain (although more or less arbitrary) standard tests for tracing the limit between the different states and for characterizing the soils by the water contents which correspond to the limits.

The most important disadvantage of Atterberg's, as of all the other "simplified" soil tests, as moisture equivalent, etc., is that their physical meaning is very complex, to such an extent that it can not possibly be expressed in terms of space, mass and time.

The complexity of the physical character of the tests makes it very difficult, if not impossible, to exclude from them certain arbitrary elements depending on the judgment of the observer. In addition to this, the relation which exists between the results of such tests and the mechanical properties of the soil (bearing capacity, pressure-moisture relations, permeability) is necessarily somewhat obscure and can not possibly be revealed except by experience. For these, and merely for these reasons, the preliminary investigations must inevitably be combined with a more elaborate physical one, whose outcome furnishes figures with a simple, well-defined meaning comparable to the data used in structural engineering.

THE PHYSICAL MEANING OF THE SIMPLIFIED TESTS

The purpose of this paper is to explain the physical meaning of the most important simplified soil tests and to evaluate both their possibilities and limitations.

The liquid limit.—It has been pointed out by Atterberg that the lower liquid limit is a rather arbitrary one, because there seems to be no definite reason why the test should be made precisely as Atterberg suggested. However, during the last several years experience has shown that the water content which corresponds to Atterberg's lower liquid limit has a singular significance. Suppose a clay or mud deposit has been formed by sedimentation in a pool or in a lake. At the beginning the deposit is liquid. However, as time passes the water content of the deposit gradually decreases and finally becomes constant. For this final state it has been found that the water content of the top layer of the deposit is approximately equal to the liquid limit.

Physically, the lower liquid limit merely means the water content at which the soil grains still have a certain degree of liberty to readjust themselves under the influence of slight vibrations without changing partners freely. For equal materials the lower limit increases with decreasing grain size (decreasing "specific grain number"). At equal grain size the lower liquid limit is the higher the more scale-like particles the soil contains. Hence, the value of the lower liquid limit may indicate different properties according to the character of the soil, and nothing can be told about its significance unless one knows the other data of the soil.

The plastic limit.—The plastic limit represents the lowest water content at which the soil can still be worked into threads with a diameter of one-eighth of an inch without breaking into pieces. The difference between the lower liquid and the plastic limit has been called by Atterberg the plasticity index. The greater the plasticity index the more plastic the material should be.

On hearing these statements one can not avoid a question as to what they mean. There can not be any doubt about the significance of the term "plasticity." As used in Atterberg's definitions, it merely means the capacity of the soil to undergo certain important changes in shape (changes involving a complete rearrangement of the particles) without a noticeable change in volume. Thus, in contrast to the clays, the sands are not plastic, because every appreciable change in shape of a mass of sand is associated with a very considerable increase in the volume of the sand.

Hence the controversial part of the statement does not concern the plasticity in itself, but the degree of plasticity. The striking difference between the attitude assumed by different investigators toward Atterberg's plasticity index is essentially due to two facts:

(*a*) Failure to define clearly the meaning of "degree of plasticity," and

(*b*) The hopeless attempt to correlate with each other the plasticity of different substances.

The term "degree of plasticity" has as much or as little meaning as has the term "degree of fluidity" of liquids. The degree of fluidity can be viewed from two different angles, according to whether one considers the limits of the range within which the substance is liquid (freezing point and boiling point) or considering the viscosity of the liquid within the range of fluidity (viscosity plotted against temperature). In a perfectly analogous way we can either consider the range of moisture within which a soil is plastic (Atterberg's method, upper and lower limit of plastic state) or we can consider the stiffness of the plastic material (ultimate compressive strength plotted against moisture content). Hence, if the problem is considered in such a way, there is no doubt left as to what the terms "plasticity" and "degree of plasticity" mean. The causes of plasticity do not even appear in these equations.

The attempt to correlate with each other the plasticity of different substances is, by necessity, hopeless because the plasticity of different substances may be due to essentially different physical causes. Thus the plasticity of metals is due to the formation of sliding planes across the crystalline grains; that of wax is due to the amorphous structure of the substance; of clays, to the presence of scalelike particles; and of fresh mortars, to some other still unknown causes. For this very reason the study of the plasticity of different materials requires different experimental methods. Thus, while Atterberg's method seems to be most satisfactory for soils, it fails, for instance, applied to the plasticity of mortars.

PHYSICAL SIGNIFICANCE OF THE PLASTIC LIMIT

According to what has preceded, the lower plastic limit represents the water content at which a soil ceases to be plastic, the term "plastic" meaning merely the capacity of the soil to be rolled out into threads with a certain standard diameter. However, experience has shown that the lower plastic limit has, for clay soils, a still deeper physical significance.

According to the writer's observations, the coefficient of permeability of a homogeneous clay soil decreases rapidly with decreasing water content, until, at the plastic limit, it becomes practically equal to zero regardless of the value of the plastic limit. Furthermore, the speed at which the water evaporates from the surface of a clay sample is about 4 per cent greater than the speed with which it evaporates from a free water surface, provided the moisture content is higher than the plastic limit. At the plastic limit the "relative speed of evaporation" starts to decrease and, for still lower moisture contents, this speed rapidly decreases with decreasing moisture content. From these and other facts it has been concluded that, for moisture contents equal to or smaller than the plastic limit, the physical properties of the water are no longer identical with those of free or of ordinary water.

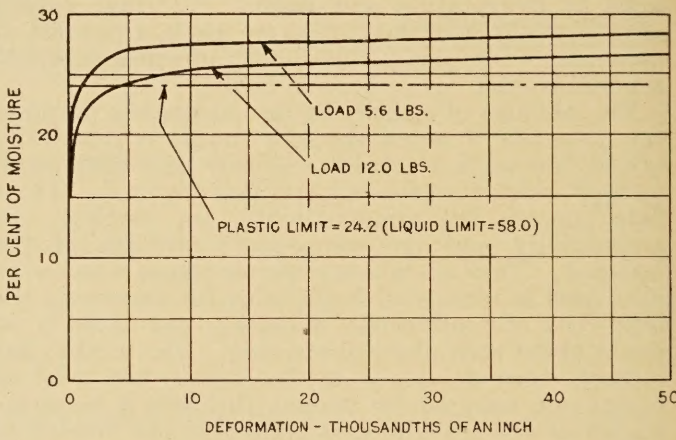

FIG. 1.—DEFORMATION DIAGRAM FOR A YELLOW BOSPHORUS CLAY SOIL SHOWING HOW THE DEFORMATION OF A CUBE OF THE SOIL PRODUCED BY A GIVEN LOAD INCREASES SHARPLY AS THE MOISTURE CONTENT PASSES THE LOWER PLASTIC LIMIT

Plasticity index (plastic and friable soils).—According to Atterberg the plasticity index is equal to the difference between the liquid and the plastic limit. The greater the plasticity index the greater is the quantity of sand with which the soil can be mixed without losing the property of being plastic. For this and for other reasons, Atterberg considered the value of the plasticity index as a standard for the degree of plasticity of the soil. According to the value of this index, he distinguished between—

Friable soils (plasticity index less than 1).
Feebly plastic soils (Index 1 to 7).
Medium plastic soils (Index 7 to 15).
Highly plastic soils (Index greater than 15).

There is no question about the validity of this classification, provided one accepts the definitions of plasticity set forth in the preceding paragraph. Due to complexity of the factors on which the phenomenon of plasticity depends, a classification based on the degree plasticity is certainly not fit for permanent purposes. However, in connection with a preliminary survey of the properties of a group of soils, it is fully satisfactory.

RELATION BETWEEN PLASTIC LIMIT AND CRITICAL BEARING POINT

Suppose we have a layer of clay, laterally confined, with a moisture content equal to the liquid limit and we put that layer under pressure so that the water squeezed out by the pressure can readily drain away. With increasing pressure the moisture content decreases. At a pressure of about 2 to 8 atmospheres (according to the character of the soil) the moisture content passes the plastic limit and the clay becomes semisolid. If at a certain pressure we interrupt the test, remove the pressure rapidly, at the same time preventing the clay from becoming resaturated, we obtain a clay-water mixture with a certain cube strength and a certain bearing capacity, both depending on the intensity of the pressure at which the test was stopped and on the character of the soil.

The relations which exist between pressure, water content, and cube strength are already so well known that we can theoretically compute what the outcome of such tests would be. Thus we may ask the following question: Suppose we reduce the water content of a soil from an initial value w_0 to a moisture content, w, less than w_0. Then we cut out of the soil a cylindrical body with a diameter of 1 inch and a height of 1 inch, and load it with a load of 12 pounds. What is the compression of this cylinder in thousandths of an inch? And what relation exists between this compression and the moisture content, w?

Let q = 12 pounds, the load acting on the cylinder,
w = the water content (in cubic centimeters per cubic, centimeter of solid material),
s = the compression of the cylinder, in inches per inch,
e = the base of the natural logarithms, and
a, c and p_0 = constants of the soil.

Based on the soil equations previously published by the writer,[1] we obtain for the compression s the formula

$$s=0.0294\frac{q}{e^{\frac{c-w}{a}}-p_0}\div 1.95\left[\frac{q}{e^{\frac{c-w}{a}}-p_0}\right]^3 \text{------} (1)$$

Figure 1 shows the relation which exists, according to this formula, between w', the moisture content in per cent of the dry weight of the sample, and s for a fat, yellow pottery clay, with a liquid limit of 58.0, a plastic limit of 24.2 and a shrinkage limit of 14.0. To substitute w' in the formula instead of w it is necessary to make use of the relation $w'=\frac{w}{n}$, in which n is the specific gravity of the material. For low moisture contents the compression of the cube is very small. However, for moisture contents above the plastic limit the compression rapidly increases with the moisture content. The sharp break in the curve corresponds approximately to the plastic limit. Since the bearing capacity of plastic soils is simply proportional to their cube strength, we would obtain for the relation between the moisture content and the penetration of a cylindrical bearing block a similar set of curves. The reason why the sharp break in the curve approximately coincides with the plastic limit is merely that, for most of the plastic soils, the pressure-moisture content curve rapidly flattens out at moisture contents below the plastic limit. It is what we may call a purely mechanical coincidence, without any deeper physical significance, but it seems to hold fairly universally.

[1] Terzaghi, Erdbaumechanik, 1925.

Mr. B. H. Levenson, of the Bureau of Public Roads, has realized the same fact by pure experience. He calls the moisture content, which corresponds to the sharp bend in the curve, the "critical bearing point," because for moisture contents above that point the compressions produced by concentrated loads rapidly increase with increasing moisture content. As a result of his tests, he has found that the "critical bearing point" approximately coincides with the plastic limit for all the soils he has tested.

In addition to this, Mr. Levenson has brought out another interesting point. The formula (1) and Figure 1 refer to a clay which does not contain any appreciable quantity of air; i. e., whose interstices are completely filled with water. In contrast to this, Mr. Levenson experimented with clays which consisted of a mixture of clay, water, and air. Such clays can be considered as accumulations of moist grains which slide on each other, wherever they touch, while for the most part the voids between the grains are filled with air. It is obvious that the bearing capacity of such materials depends largely on the manner in which the samples have been prepared. Nevertheless, for such samples, too, the critical bearing point is found to coincide more or less with the plastic limit.

Mr. Levenson prepared his specimen in the following way: For compression tests with unconfined soil cylinders (a) he dried 100 grams of soil, powdered the sample, and passed it through a 2-millimeter sieve. The fraction which passed the sieve was mixed with the moisture he wanted to add; the mixture of water and clay was left over night in the humidifier, and the specimen was moulded the following morning under a pressure of 70 pounds per square inch, acting during a period of two minutes. The specimen had a height of 1 inch and a cross-section area of 1 square inch. During the compression tests the top surface of the specimen was covered with an aluminum bearing block with a diameter of 1⅛ inches and a thickness of one-eighth inch.

UNCONFINED TESTS

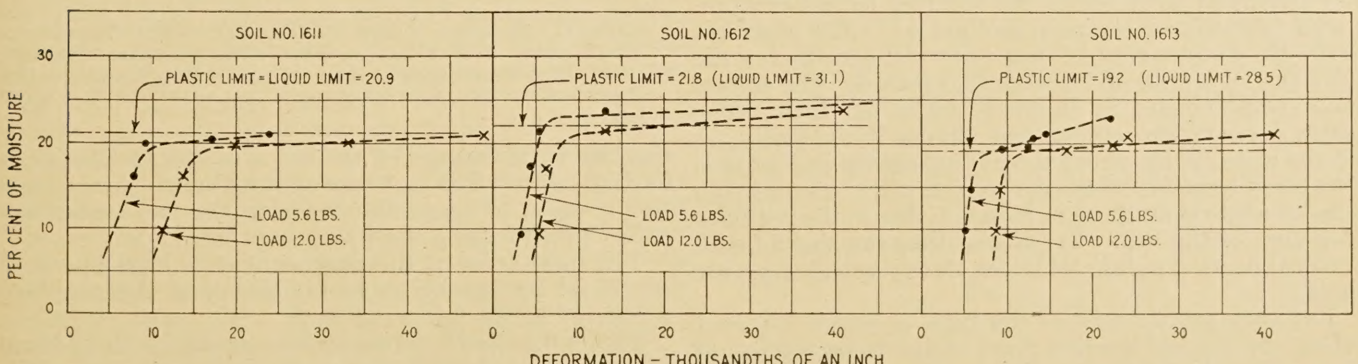

Fig. 2.—Deformation Diagrams for Three Medium Plastic Soils Tested at Arlington. The Curves Show the Deformation of Cubes of the Soils Under Pressures of 5.6 Pounds, Respectively, With Varying Moisture Content. Note That the Sharp Breaks in the Curves (Critical Bearing Points) Coincide Closely With the Lower Plastic Limits

The tests with plastic soils confined within a rigid ring (*b*) required about 300 grams of dried soil. Compacting of the sample was accomplished by a pressure of 25.1 pounds per square inch (1.76 kg. per sq. cm.), acting during a period of three minutes. The ring had a diameter of 2¼ inches and a height of 1 inch. The test was performed by placing on top of the speci-

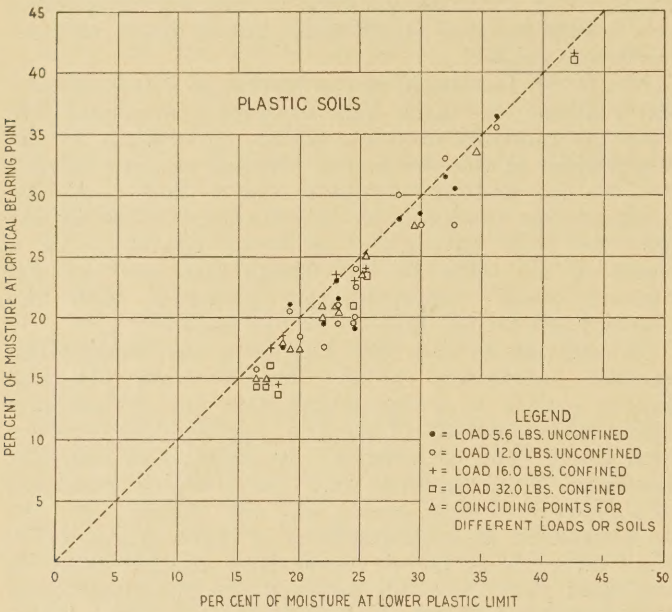

FIG. 3.—RELATION BETWEEN THE PLASTIC LIMIT AND THE CRITICAL BEARING POINT OF PLASTIC SOILS TESTED AT ARLINGTON. FOR MEDIUM AND FEEBLY PLASTIC SOILS THE CRITICAL BEARING POINT IS EQUAL TO OR SOMEWHAT LOWER THAN THE PLASTIC LIMIT

men a circular bearing block covering an area of 1 square inch and applying the load on top of this block.

The pressure used for preparing the plastic soils for the confined tests (*b*) is somewhat smaller than the pressure required for reducing the water content of an immersed specimen to the plastic limit. Hence for moisture contents above the plastic limit, the process furnished homogeneous samples, practically identical with those referred to in Figure 1. On the other hand, for moisture contents below the plastic limit the experimenter obtained what we may call a conglomerate of wet grains, which can not be compared with those whose properties are described by Figure 1.

Some of the results obtained by Mr. Levenson are represented in Figure 2. For this figure the ordinates represent the moisture (in per cent of the weight of the dry material) with which the samples have been prepared (unconfined samples, method a). The abscissae give the compression of the soil cylinders under the influence of a load of 5.6 and 12.0 pounds per square inch, respectively, in thousandths of an inch. The upper parts of the curves run almost horizontally, as do the upper parts of the theoretical curves in Figure 1. The steep branches of the curves rise under a very much smaller angle than those in Figure 1, due to the granular structure of the soil. In spite of that the sharp break (critical bearing point) coincides closely with the plastic limit.

For very plastic soils of the type to which Figure 1 refers the critical bearing point seems to be somewhat above the plastic limit. For medium and for feebly plastic soils the critical bearing point is equal to or somewhat lower than the plastic limit (fig. 3). For friable soils the plastic and the liquid limit are identical, and their critical bearing point is approximately equal to 75 per cent of the water content which corresponds to the lower liquid limit. (See fig. 4.) Hence, if passing from the very plastic down to the very friable soils, the critical bearing point gradually shifts from the plastic range into the semisolid range.

Figure 5 illustrates the effect of a sand admixture on the bearing properties of a plastic soil. Figure 5a corresponds to a plastic soil, Figure 5b represents the same soil mixed with 50 per cent of sand (pure quartz sand, passing the 100-mesh screen), and Figure 5c corresponds to the soil mixed with 90 per cent of sand. Since an admixture of sand invariably lowers the plastic limit, the critical bearing point also goes down with increasing sand content.

THE SHRINKAGE LIMIT

If the water content of a soil decreases on account of evaporation to values below the lower plastic limit, the soil is said to pass from the plastic into the semisolid state. During this process the decrease in volume of the soil is precisely equal to the value of the water lost by evaporation. However, as soon as the moisture content reaches a certain minimum limiting value the volume of the sample ceases to diminish, while the weight of the sample still continues to decrease. We say that the sample has passed from the semisolid into the solid state. The limit between the two states is

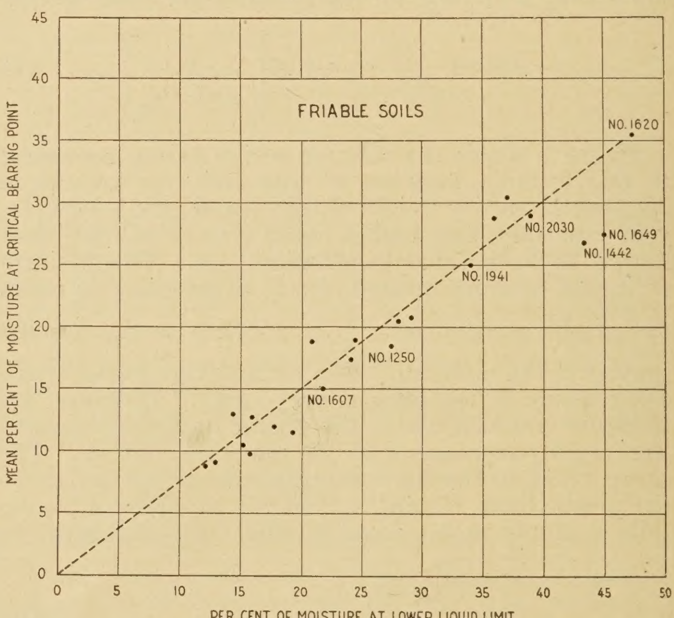

FIG. 4.—RELATION BETWEEN THE LOWER LIQUID LIMIT AND CRITICAL BEARING POINT OF FRIABLE SOILS TESTED AT ARLINGTON. FOR FRIABLE SOILS THE CRITICAL BEARING POINT IS APPROXIMATELY EQUAL TO 75 PER CENT OF THE WATER CONTENT CORRESPONDING TO THE LOWER LIQUID LIMIT

marked by a change of the color of the sample from dark to light. The moisture content which corresponds to the point of transition between the two states was called by Atterberg the "shrinkage limit."

The mechanics of the process of shrinkage has been explained by the writer in his theory of the capillary pressure [2] in the following way:

Figure 6A shows a cross section through a small lump of soil whose moisture content is greater than the shrinkage limit. The voids are completely filled with water. The free surface of the water is located within

[2] Terzaghi, Erdbaumechanik. 1925.

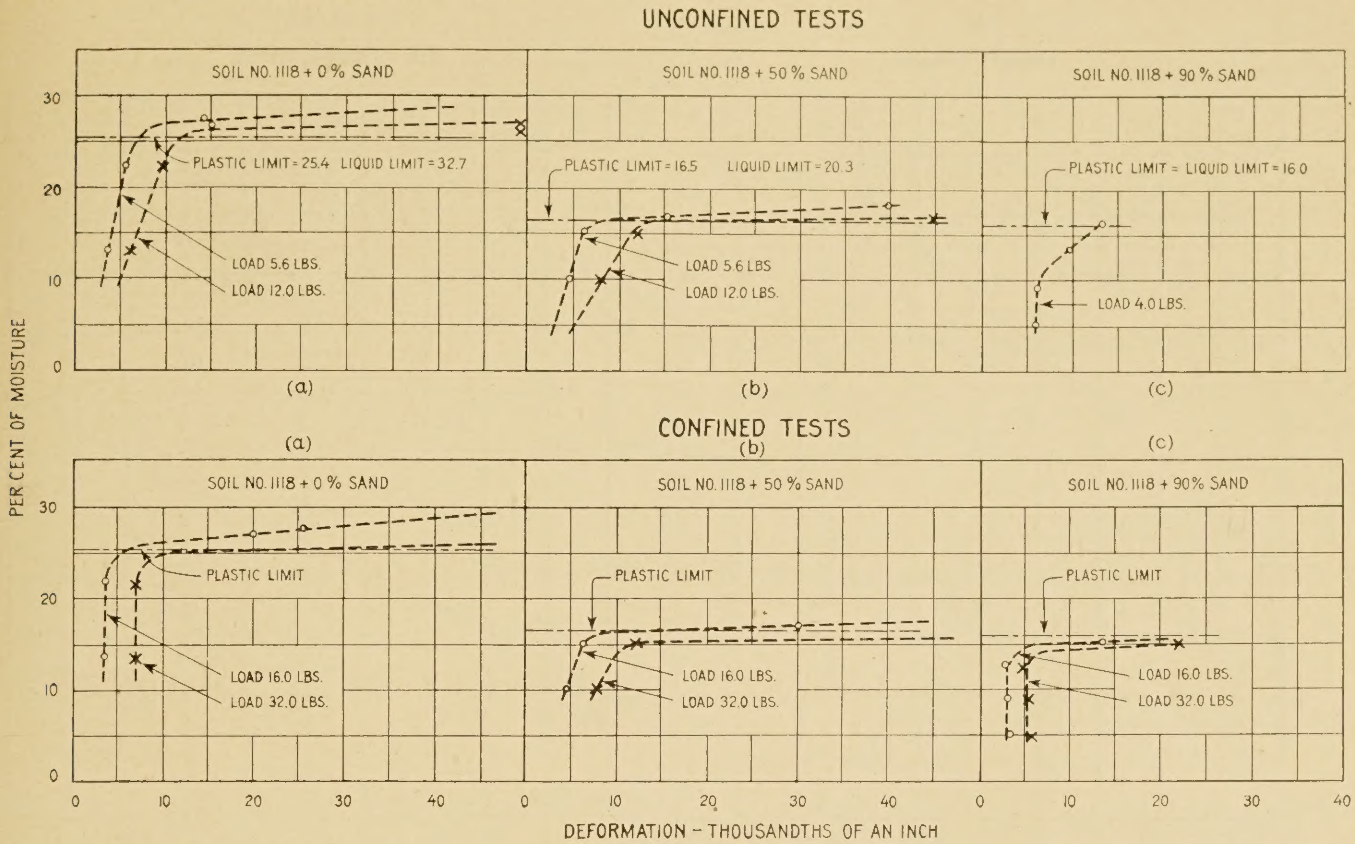

Fig. 5.—Relation of Moisture Content and Deformation of a Soil Tested at Arlington, Showing the Effect of Admixtures of Sand on the Bearing Properties of the Soil

the surface of the sample and the surface tension exerts on the outer surface of the sample a uniformly distributed pressure, in every respect comparable to an external pressure acting like a hydrostatic pressure at every point perpendicular to the outer surface of the sample. This surface tension, which causes the soil to shrink, is the same force which drives the water up in a capillary tube and which prevents the water from flowing back through such a tube, provided the length of the tube is smaller than the height of capillary rise. No air can get from outside into such a tube and, for the same reason, no air can get into the clay as long as the pressure exerted by the surface tension is smaller than the pressure required to still further compress the clay.

At the lower liquid limit the pressure exerted by the surface tension of the water or by the "capillary pressure," as it should be called, is practically equal to zero. At the lower plastic limit it amounts already to several atmospheres and, while the sample passes through the semisolid state, the capillary pressure becomes still higher. Finally, however, there comes a point where the force required to produce a volume change equal to the volume of the evaporating water becomes greater than the maximum value the capillary pressure can possibly assume. Hence, if still more water evaporates, the surface of the capillary water withdraws into the interior of the soil. Air enters the voids and, as a consequence, the color of the soil changes from dark to light. The water content at which this change occurs is the "shrinkage limit." The shrinkage limit obviously depends on two factors: The compressibility of the soil and the maximum value of the capillary pressure.

Due to the simple geometrical relation which exists between the volume change and the water content, the shrinkage limit S can easily be computed from the following data: Volume, V, of the sample at water content, w, the volume, V_0, of the dry sample (identical with the volume of the soil at the shrinkage limit) and

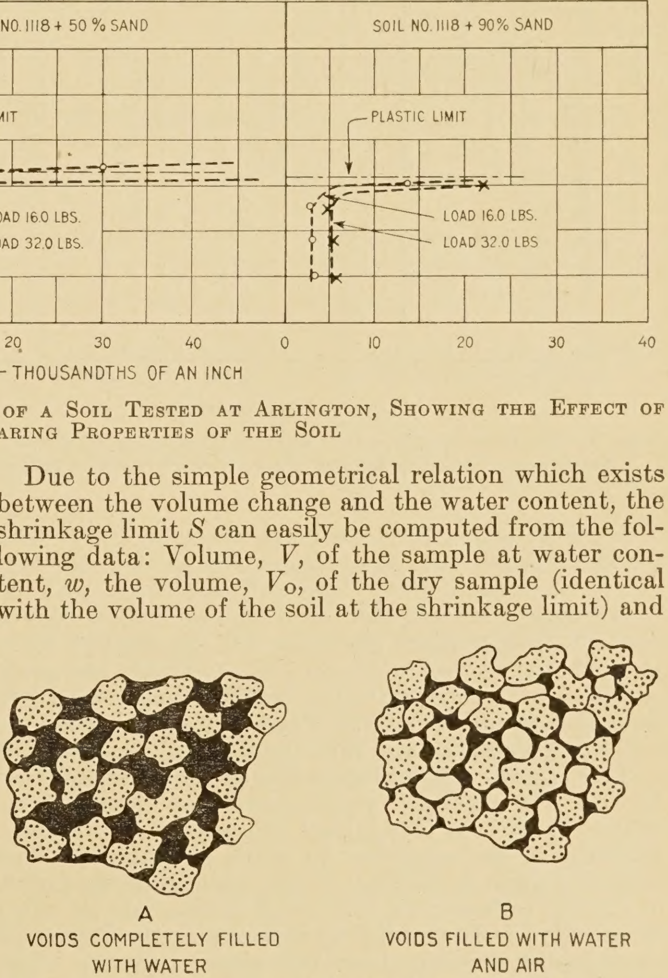

Fig. 6.—Cross Sections of a Lump of Soil, (A) When the Moisture Exceeds the Shrinkage Limit, and (B) When the Moisture Content Is Less Than the Shrinkage Limit

the weight, W, of the dry sample. At water contents above the shrinkage limit no air gets into the sample. Hence the difference, $w-S$, between the original water content, w, and the shrinkage limit, S, is obviously equal to the volume change, $V-V_0$, divided by the dry weight, W, or

$$\frac{V-V_0}{W}=w-S$$

and therefore

$$S=w-\frac{V-V_0}{W}$$

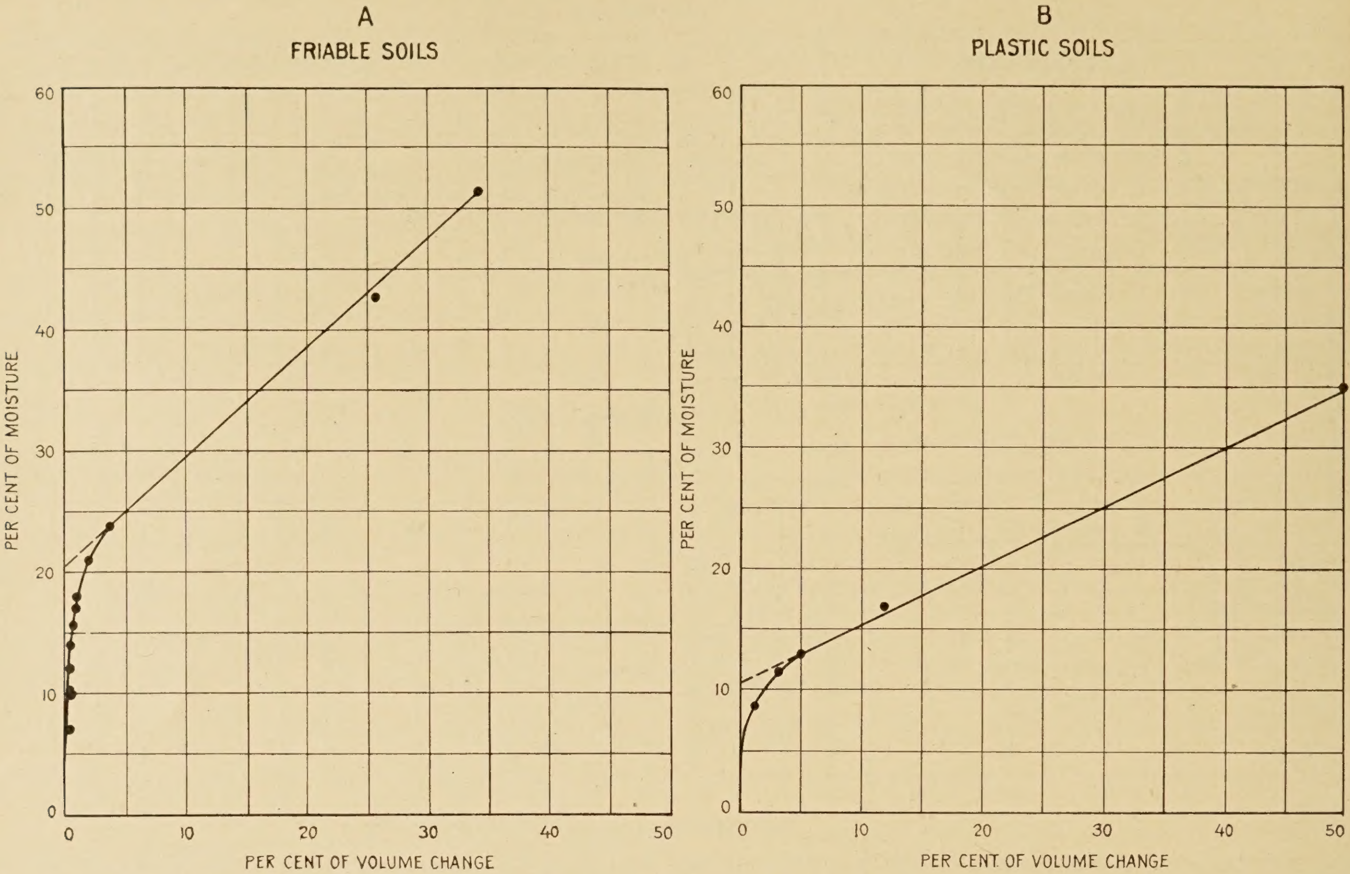

FIG. 7.—TYPICAL RELATION BETWEEN MOISTURE CONTENT AND VOLUME CHANGE FOR FRIABLE AND PLASTIC SOILS. THE SHRINKAGE LIMIT IS INDICATED BY THE INTERSECTION OF THE EXTENDED PORTION OF THE STRAIGHT PART OF THE CURVE WITH THE VERTICAL AXIS

If this equation be plotted as a diagram, one obtains for the relation between volume change and moisture content a straight line. The ordinate of the point of intersection between this line and the vertical axis is equal to the shrinkage limit (fig. 7). In practice, however, the semisolid state passes not suddenly but gradually into the solid state for the following reason: As soon as the surface of the water withdraws from the outer surface of the soil into the interior (fig. 6B), the surface tension acts no longer like an external pressure, because the capillary water forms now innumerable droplets scattered throughout the interior of the soils, separated from each other by small air bubbles. The surface tension is confined to the vicinity of the points of contact between the individual grains, and its effect essentially consists in slightly rearranging the individual particles by pulling them still closer together. The additional compression produced by this process is very small. Nevertheless, it shows up quite clearly on every moisture content—volume change diagram. (See fig. 7.)

RELATIONS BETWEEN THE LIMITS OF CONSISTENCY

From the preceding discussion of Atterberg's limits it became evident that each one of the limits depends on different factors cooperating in a different way in producing the consistency to which the limits refer. Thus the lower liquid limit essentially depends on the number of grains per unit of volume and, to a smaller extent, on the shape of the particles. On the other hand, the plastic limit and the index of plasticity largely depend on the shape of the grains while the number of grains (specific grain number) seems to play a less important part. The shrinkage limit varies with both factors and, in addition, to a large extent with the uniformity of the soil. Hence it is obvious that both the values of the limits and the relation between the different limits are apt to vary with the soil (fig. 8). In general, for friable soils, the shrinkage limit may be anywhere between the lower liquid limit and 50 per cent of its value. For plastic soils the shrinkage limit ranges between a water content of 10 and 20 per cent (in exceptional cases more), and no definite relation exists between the plastic and the shrinkage limit.

If a plastic soil is mixed with sand, both the lower liquid and the lower plastic limit decrease with increasing sand content. At the same time the difference between the two limits (Atterberg's plasticity index) decreases and, at a certain percentage, p, of sand, the plasticity index becomes equal to zero. According to Atterberg, p should be the higher, the greater the plasticity index.

THE MEANING OF THE MOISTURE EQUIVALENT

The limits of consistency described in the preceding paragraphs have always the same physical significance regardless of the fineness of the soil. In contrast to this an analysis of the moisture equivalent test has shown that the meaning of this test essentially depends on the effective size of the soil grains.

This analysis was based on the known relation which exists between the centrifugal force, the effective size of the voids, and the maximum value of the surface tension. It led to the following conclusions:

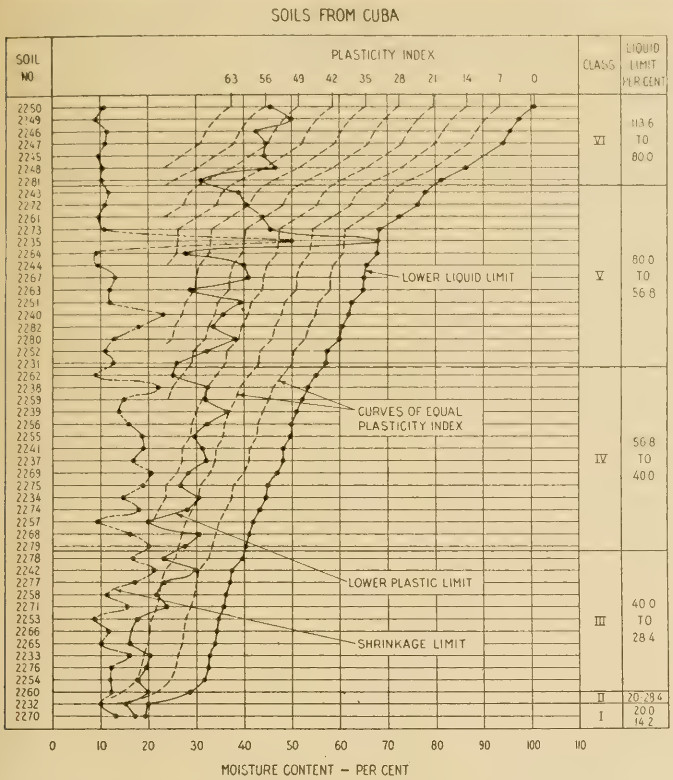

Fig. 8.—Lower Liquid and Plastic Limits and Shrinkage Limits of Soils from Cuba Tested at Arlington, Showing by the Length of Abscissa Between the Plastic and Liquid Limits the Plasticity Index of the Soils

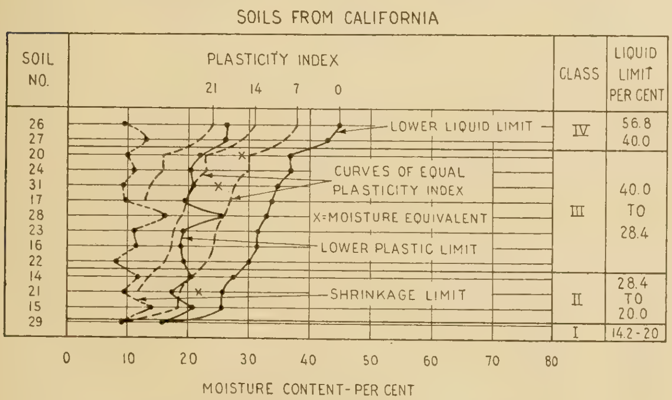

Fig. 9.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from California

(*a*) If plastic soils (ordinary clay soils) are submitted to the moisture equivalent test, the voids of the soil remain completely filled with water. The centrifugal force combined with the surface tension of the water exerts on the sample a force whose intensity depends on the speed of rotation, on the thickness of the layer of soil, and, for very plastic soils, on the duration of the test. The greater the speed of rotation the thicker the layer, and (for very plastic soils) the greater the duration of the test the smaller is the moisture equivalent. If performed under the standard condition described in Public Roads, April, 1925,[3] the moisture equivalent represents the water content which corresponds to an external pressure of approximately 2 kilograms per square centimeter, acting for a period of one hour. Since the pressure required for reducing

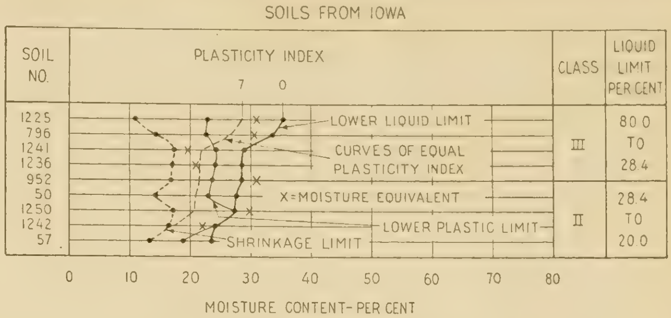

Fig. 10.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from Iowa

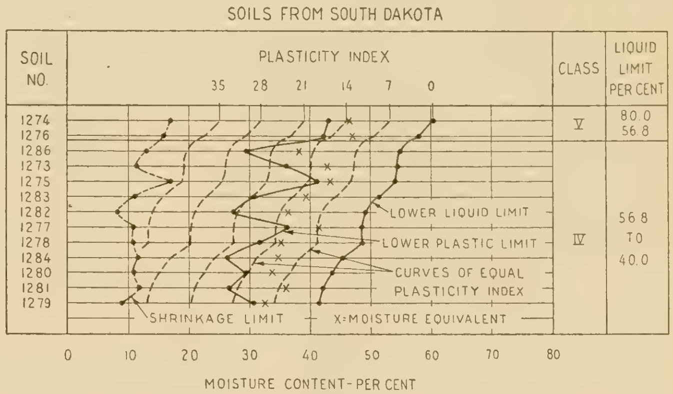

Fig. 11.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from South Dakota

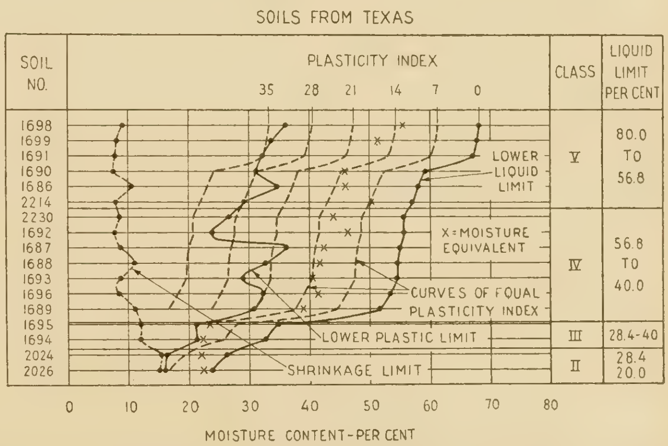

Fig. 12.—Regional Diagram Showing Limits and Moisture Equivalent of Soils from Texas

the water content of a clay down to the lower plastic limit amounts to 2.5 kilograms per square centimeter or more, the moisture equivalent for plastic soils should always be somewhat higher than the lower plastic limit. Figures 9 to 12, inclusive, show the limits and the moisture equivalent for a great number of soils tested in the experimental station of the Bureau of Public Roads at Arlington. For all the plastic soils (i. e., for all those soils whose lower liquid limit is appreciably higher than the lower plastic limit represented on the diagrams) the moisture equivalent is located between the two limits.

Hence from the results of our analysis it became evident that, for plastic soils, the moisture equivalent has nothing whatsoever to do with the drainage properties of the soil. It merely tells about the compression produced by a certain pressure.

[3] Procedure for Testing Subgrade Soils, by J. R. Boyd, Public Roads, vol. 6, No. 2, April, 1925.

(*b*) If friable soils with fairly large voids are submitted to the moisture equivalent test, the centrifugal force acting on the capillary water overcomes the surface tension. Part of the capillary water is ejected from the voids; air invades the interior of the sample and the capillary water is split up into a great number of small droplets, adhering to the points of contact between the individual grains. As a consequence, the pressure acting on the sample during the test has no more a definite value. The pressure depends on the ultimate water content which, in turn, depends on the number of points of contact per unit of volume. The greater this number the greater the moisture equivalent. Hence in this case, the moisture equivalent becomes an index of the draining properties of the soil. Since not all of the voids are filled with water, the moisture equivalent should, for friable soils, be lower than the lower liquid limit. This statement, too, is confirmed by the data represented by Figures 9 to 12, inclusive.

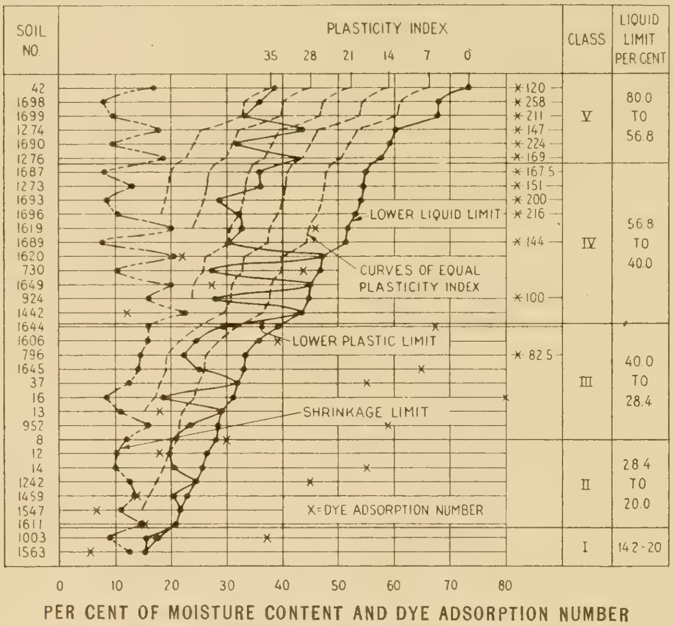

Fig. 13.—Regional Diagram Showing Limits and Dye Adsorption Numbers of Soils

(*c*) For soils intermediate between plastic and friable soils the moisture equivalent has no definite meaning at all.

FIELD MOISTURE EQUIVALENT TEST ERRONEOUSLY NAMED

In connection with the earlier attempts to investigate the properties of subgrades, A. C. Rose, of the Bureau of Public Roads, developed a field test which was believed to furnish the moisture equivalent without using the centrifuge. He described his test as follows:[4] "The test is made by taking a 500-gram sample of air-dried soil, breaking up the lumps, placing the sample in a bowl, adding water slowly from a burette, mixing the water and soil until it reaches the consistency of putty and may be compacted with a spoon or spatula without any free water remaining on the surface. Water is then allowed to drop upon the smoothed surface as long as it is abosrbed. Before the moisture equivalent percentage is reached, the sample will absorb water readily, but when the critical value is passed, the surface will retain a wet, shining appearance."

From the mere description of the test one learns that the result has physically nothing in common with the result of the standard moisture equivalent test. For plastic soils the moisture equivalent represents the moisture which corresponds to a pressure of approximately 2 kilograms per square centimeter, provided the initial moisture content of the sample was considerably greater. In contrast to this, Mr. Rose's field-moisture determination represents the water content which corresponds to a pressure of zero, provided the initial water content of the sample was considerably lower.

In order to realize the physical meaning of the field-moisture test, one has to remember the fact that under normal conditions the capillary water contained in the voids of a plastic soil with a plastic consistency is in a state of tension. Evaporation of capillary water increases the tension, while the moistening of the sample causes the tension to drop. (See the paragraph on the shrinkage limit.) Tension in the capillary water means a negative hydrostatic pressure. On the surface of the sample, the hydrostatic pressure is equal to zero. Hence, if one places a drop of water on the surface of a piece of soil whose capillary water is in a state of tension, the water flows at once toward the zone of negative pressure, and one says the water "has been absorbed". However, by steadily increasing the moisture content one steadily decreases the tension in the capillary water until finally a point is reached where this tension becomes equal to zero. This point obviously represents Mr. Rose's field moisture, because the criterion of the test consists in the sample losing its property of absorbing additional moisture.

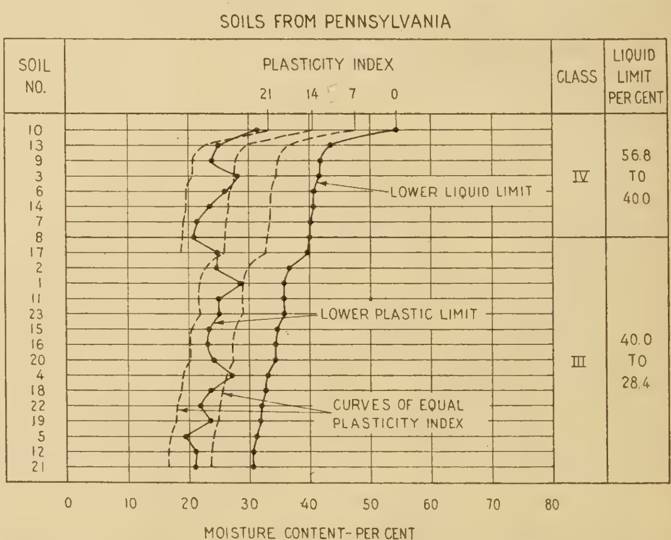

Fig. 14.—Regional Diagram Showing Limits of Soils from Pennsylvania

Figure 15 shows the relation between pressure and voids ratio for a feebly plastic Pennsylvania soil. The initial water content of the sample was equal to the lower liquid limit. By gradually raising the pressure from zero to 3 kilograms per square centimeter (AC), the voids ratio dropped from 0.837 to 0.595. By reducing the pressure again from 3 kilograms per square centimeter to zero, the voids ratio increased again from 0.595 to 0.625 (rebound curve CD).

[4] Practical Tests for Subgrade Soils, by A. C. Rose, Public Roads, August, 1924.

In the diagram, the section, AB, of the upper curve represents the changes the sample would have undergone during a moisture equivalent test (raising the pressure from zero to 2 kilograms per square centimeter), and the ordinate of the point, B, represents the moisture equivalent in terms of voids ratio. In contrast to this, the field-moisture test is represented, not by a part of the upper but of the lower curve CD, and the ordinate of the point D represents the field moisture. In this particular case the "field-moisture" test would have furnished practically the same result as the moisture equivalent test (0.625 against 0.622 in terms of voids ratio). Nevertheless, the close agreement between the values would have been no less a mere mechanical coincidence than is the coincidence between the plastic limit and the critical bearing point.

Thus the "field-moisture" determination merely indicates the maximum amount of moisture a soil is apt to absorb when its moisture content is increased from a lower value by gradually adding moisture. The objectionable feature of the test consists in the fact that the "field moisture" obviously depends on the initial water content. Thus, if the test is started at the point B, of the diagram, Figure 15, instead of at point C, the sample would have passed through the process BD[1], and the result of the test would have been a voids ratio of 0.661 instead of 0.625. For this reason one can hardly expect consistent results in case the tests are made by two independent observers.

According to Mr. Rose, the soils of the Pacific Northwest were found to represent good, mediocre, or poor subgrades, according to whether their "field moisture equivalent" was smaller than 20, from 20 to 30, or greater than 30 per cent. Moisture contents of 20 and 30 per cent, respectively, correspond to the volumes of voids of 35 and 44 per cent, respectively. Considering this fact and the physical meaning of the field moisture test, Mr. Rose's statement has the following significance: A subgrade is good or mediocre according to whether a thorough soaking after a period of dryness does not increase its volume of voids beyond 35 or 44 per cent, respectively. This experience is closely related to the fact that the same sand may have a very great bearing capacity or almost none according to whether its volume of voids is equal to 42 or 48 per cent[5] (more or less, depending upon the nature of the sand). This, in turn, is connected with the fact that the volume of voids in an accumulation of equal spheres ranges between the extreme limits, 25.8 per cent and 47.6 per cent. The greater the volume of voids, the smaller the resistance of the spheres against mutual displacement, and beyond a value of 47.6 per cent the accumulation is unstable, which means that its bearing capacity drops to a very small value.

The "ultimate swell," represented by Mr. Rose's field moisture, is among the data furnished by the standard swelling test which has recently been incorporated in the investigation program of the Bureau of Public Roads. Figure 15 represents the results of such a test. Since both the initial state of the sample and the pressure corresponding to the point, C, are standardized, the personal element is excluded and the "ultimate swell" thus obtained represents a physical fact with known meaning. It will be among the problems of the proposed field survey to determine whether the ultimate swell is the only factor which governs the behavior of the subgrade or whether there are other additional factors entering into the relation between subgrade and road surface. For the time being it would be premature to express any opinion concerning the possible outcome of this investigation. However, Mr. Rose's observations represent a very valuable group of facts to start with.

THE DYE ADSORPTION TEST

The results of all those tests which have been described in the preceding paragraphs essentially depend on the mechanical properties of the soil; i. e., on the size and shape of the grains and on the uniformity of the mixture. In contrast to this, the result of the dye

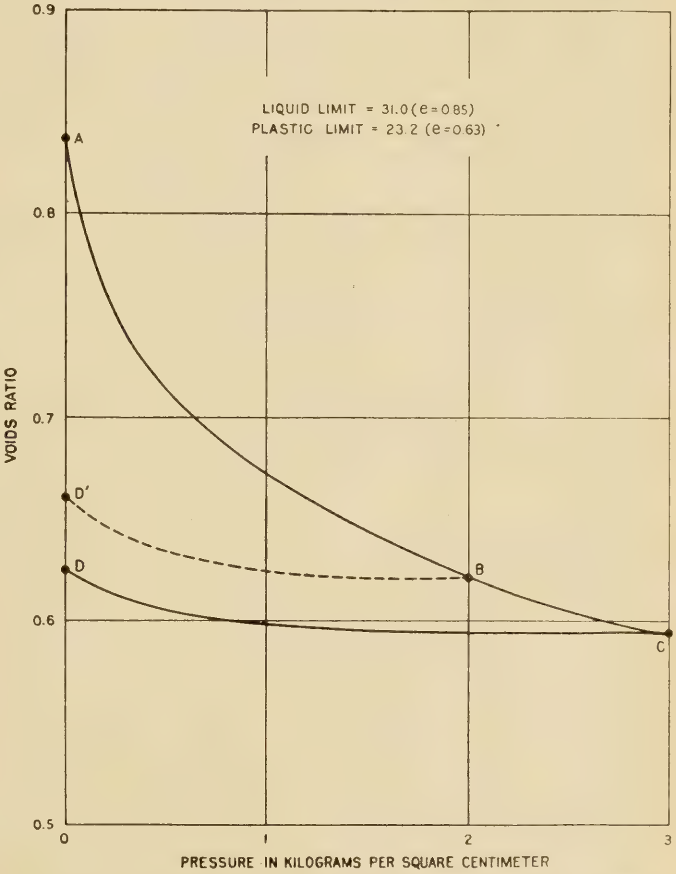

Fig. 15.—Pressure-Voids Ratio Diagram for a Feebly Plastic Pennsylvania Soil, Illustrating the Difference Between the True Moisture Equivalent (B) and the So-Called "Field Moisture Equivalent" (D, D′)

adsorption test seems to depend essentially on the chemical nature of the soil particles and on their electrolyte content. As a consequence, there seems to be no definite relationship between the limits and the dye adsorption number, except for the fact that the soils with high liquid limits have higher dye adsorption numbers than soils with a low liquid limit. To illustrate this statement, Figure 13 may serve. In this figure, the erratic character of the dye adsorption number is quite conspicuous, soils with similar limits having widely different dye adsorption numbers. Hence it is obvious that the dye adsorption number expresses among others a factor independent of those which determine the limits. This, in itself, is a very valuable property of the dye adsorption test. However, before we are able to make any statement concerning the value

[5] Terzaghi, Erdbaumechanik, 1925.

of the dye adsorption test, we must know whether the factor expressed by the dye adsorption number has an appreciable effect on the mechanical properties of the soil. This question can not possibly be decided except by a systematic physical investigation of soils with similar limits, but with widely different dye adsorption numbers.

INTERPRETATION OF TEST RESULTS

If dealing with the interpretation of test results obtained in the laboratory, we must first of all consider two important facts. The first one consists in the soil tests being confined to that fraction of the soil which passes a certain standard mesh screen (width of opening 0.5 mm.). Hence a correction must be made for the influence of the coarser constituents on the properties of the soil. For the time being, no reliable basis for making such a correction exists and systematic investigations will be required for establishing such a basis.

The second fact consists in the structure of the undisturbed soil being different from the structure of the soil subjected to the tests in the laboratory. It is conceivable that the same soil may exhibit somewhat different properties according to whether the structure of the top layer has developed under the influence of a very cold or a very warm, a humid or an arid climate. It is one of the problems of the field survey to investigate the importance of these influences.

These two facts impose upon the interpretation of the test results obtained in the laboratory certain unavoidable limitations. The man working in the laboratory can not possibly do more than furnish the data required for identifying the raw materials of which the subgrades consist. Hence all his efforts ought to be governed by this and merely by this one intention.

Since the sole function of the subgrade consists in supporting the road surface, we are merely interested in its mechanical properties; that means in its compressive strength, its elastic properties (swelling), its permeability and in the influence of the moisture content on its resistance. The physical meaning of these properties is as well defined as is the meaning of the data used in structural engineering.

In contrast to this, the simplified soil tests have apparently no bearing on the strength of the materials except for the fact that the plastic limit approximately coincides with the "critical bearing point." Hence we are entitled to ask wherein their value resides. The answer to this question can easily be deduced from the preceding analysis of the meaning of the simplified soil tests. According to this analysis, the result of the simplified soil tests depends precisely on the same physical factors which determine the resistance and the permeability of the soils (shape of particles, effective size, uniformity) only in a far more complex manner. Thus for instance a high lower liquid limit associated with a low shrinkage limit may mean either an excessively fine-grained soil with a low swelling capacity or a more coarse-grained soil with a very marked swelling capacity. According to whether these two limits have the first or the second meaning, the plastic limit would be higher or lower. In spite of these complications, we are safe in making the following statement: If several soils with similar geological origin have fairly identical limits, their physical properties too will be fairly identical and it will be sufficient to investigate a single one of them more thoroughly. Hence the limits represent an excellent means for preliminary soil classification.

If we get the mere description of the appearance of a soil, we know about that soil practically nothing, and an identification is simply impossible. If we know the three limits of the soil, we are already in a position to compare this soil with others and can at least anticipate what its properties may be. If we know in addition the results of physical tests performed on another soil with fairly identical limits, we can say the soil is known.

REGIONAL SOIL DIAGRAMS FACILITATE CLASSIFICATION

The tabulated results of the determination of the limits for a group of soils consist of a confusingly great number of apparently incoherent figures, and it is very difficult to perceive the relationship which exists between the soils represented in the table. In order to facilitate this work, and to visualize the characteristic features of the soils, it is proposed to represent the results of the investigation in a graphical form. For this purpose it is advisable, first of all, to arrange the soils according to the value of the lower liquid limit, because this limit is by far the most variable one, ranging anywhere between 15 and more than 100 per cent.

The graphical representation is performed in the following manner: The individual soils are represented by horizontal lines, one quarter of an inch apart. On every line the limits of the soil are plotted to a scale of 1 inch equal to 10 per cent moisture content. Then all the points representing the same limit are connected with each other by continuous curves. Since the soils have previously been arranged according to the value of their lower liquid limit, the lower liquid limit curve slopes in one direction only, while the others have a more or less sinusoidal shape. The result of this operation will be called the regional soil diagram, representing the variation of the limits of soils coming from a definite region.

By tracing curves parallel to the lower liquid limit curve at horizontal distances representing 7, 15, 22 per cent, etc., from this curve, one divides the space between the liquid limit and the plastic limit curve into sections of uniform width. Since the total distance between the liquid and the plastic limit represents the plasticity index, the subdivision of this distance by parallel curves facilitates the evaluation of the degree of plasticity (according to Atterberg's definition) of the soils.

Figures 8 to 12 inclusive and Figure 14 represent the regional soil diagrams for soils from Cuba, California, Iowa, South Dakota, Texas, and Pennsylvania investigated in the laboratory of the Bureau of Public Roads at Arlington. By merely glancing at these diagrams one sees at once the difference which exists between the soils coming from the different regions. For the time being, we do not precisely know wherein the significance of the limits—for instance, the significance of a low plastic and a high liquid limit—resides, but the mere fact that there is a great difference between the average limits of the soils coming from different sections of the country plainly shows that there must also be a very considerable difference between their physical characteristics. The knowledge of the significance of the limits will automatically come as soon as a sufficient number of samples has been submitted to the standard physical tests.

(Continued on p. 170)

DIRECT PRODUCTION COSTS OF BROKEN STONE

By DR. GEORGE E. LADD, Economic Geologist, Bureau of Public Roads

THERE are no available sources of information as to the direct costs of producing broken stone which are satisfactory for comparative purposes by quarry operators. Something can be found in cost data manuals on certain operations but the material is scattered and unsatisfactory because the conditions under which the operations were conducted are usually not stated.

Some time ago the Bureau of Public Roads undertook a study of quarries and broken stone production, especially direct costs and conditions governing them. It was decided not to enter the field of overhead costs which vary largely at different quarries and in different localities, and the following items also were omitted from the study: Capital investment, interest, royalties, general repairs, insurance, depreciation, depletion, opening of quarry face, general supervision, accounts, sales, and delivery. This reduced the investigation to a study of the direct production costs which included only such supervision as that employed directly in quarry or plant.

Different types of quarries, pit and open-face, high and low-face, and large and small plants operating in various kinds of rock were selected for study and all conditions were noted that might affect costs. As it was desired to make the results of practical value at any time, quarry methods of operation, wage rates, costs of materials and labor hours, as well as dollar costs per unit were recorded. Since the results are given not only in terms of dollars but also in terms of labor hours and materials they should be of permanent value to quarry operators and all others engaged in rock excavation.

METHOD OF STUDY

The operation of producing broken stone was divided into 10 natural units as follows: Stripping, drilling face, drilling bowlders, blasting face, blasting bowlders, sledging and steel balling, pumping, loading and hauling waste; loading and delivery to crusher; and crushing, screening, etc.

The data were acquired by daily visits to selected quarries for a period usually of eight weeks. Pay rolls and books, where kept, were consulted and all results were checked by personal observation. Methods of operation were studied and the rock being quarried was sampled for laboratory test to determine its nature.

A complete report on these studies, giving a description of each of the 23 quarries studied with data collected at each and comparisons of operations at the various quarries, is to be published but it is considered worth while to present at this time some of the comparisons and conclusions. The various items will be discussed in the order listed above. In studying the data presented in the different tables it may be desirable to know the scale of operation of the different quarries which are referred to by key numbers, and this information is given in Table 7.

DISCUSSION OF OPERATING COSTS

Stripping.—Stripping was conducted at 14 of the 23 quarries. At the others there was either no stripping or so little that it was shot down with the face and eliminated in the screening operation. At the 14 quarries where stripping was done the cost varied from $0.0022 to $0.0873 per ton of broken stone. With one exception all stripping was removed by hand labor and dump carts.

Drilling face.—Table 1 summarizes the data on face drilling. It should be noted that low cost per foot of drilling does not necessarily mean low cost of drilling face per ton of broken stone on account of wide differences in the spacing and depth of holes, and the combination of snake holing with vertical holes.

FIG. 1.—THE UPPER PICTURE ILLUSTRATES THE ADVANTAGE OF A LEDGE (TRAP) THAT BLASTS INTO RELATIVELY SMALL BOWLDERS, WHILE THE LOWER ONE SHOWS A GRANITE LEDGE THAT BREAKS INTO LARGE BOWLDERS. SUBSEQUENT BOWLDER BREAKING COST 1½ CENTS AT THE FIRST QUARRY AND 12 CENTS AT THE SECOND

The average rate of drilling per hour was 2.4 feet for tripod drills and 1.46 feet for well drills. Comparison of rates of drilling with the laboratory results of the wear test indicates a relation between them which

TABLE 1.—*Drilling face data* [1]

Quarry key No.	Kind of rock	Kind of drill	Direct cost of drilling face per ton of broken stone	Cost per foot drilled	Average feet drilled per hour per drill	Tons down per foot drilled	Factors affecting results
18	Diabase (trap)	Tripod	$0.0247	$0.80	1.81	69.09	High face, numerous joint planes, tripods, snake holes, electricity, air.
8	Dolomite	Well	.0250	.68	1.29	27.24	High face, well drills, deep holes, far apart, electric power.
[2] 17	Diabase (trap)	do	.0264	2.06	.65	53.47	High face, well drills, hard rock, deep holes, far apart, steam power.
21	Limestone	do	.0273	.53	3.10	19.23	Medium high face, well drills, soft rock, holes far apart, electric power.
20	Dolomitic limestone	do	.0308	.74	2.48	24.53	High face, well drills, soft rock, holes far apart, gasolene power.
7	Siliceous dolomite	do	.0375	.94	.80	25.15	Do.
22	Marble	Tripod	.0470	.40	(3)	8.51	High face, tripods, soft rock, snake holes, electric power, air.
19	Diabase (trap)	do	.0487	1.57	1.61	32.25	High face, tripods, hard rock, snake holes, electric power, air.
23	Slate	Well	.0596	1.43	.91	24.09	High face, well drills, hard rock, snake holes, steam power.
3	Granite	do	.0819	1.83	1.68	18.70	High face, well drills and tripods, hard rock, oil and electricity.
10	Diorite (trap)	Tripod and well.	.1167	.76	1.20	6.48	High face, well drills and tripod (mostly), hard rock, electricity, air.
12	Altered rhyolite (trap)	Tripod	.1323	.69	2.34	5.19	Low face, benches, medium hard rock, hole spacing close, electricity, air.
15	Trachytic rhyolite (trap)	do	.1835	.42	4.13	2.31	Low face, benches, tripod, medium hard rock, hole spacing close, steam.
6	Conglomerate	do	.1845	.45	4.39	2.45	Low face, benches, tripod, medium hard rock, hole spacing close, electricity, air.
14	do	do	.2382	.95	2.54	4.00	Low face, benches, tripod, medium hard rock, hole spacing close, steam.
1	Granite	do	.2384	.87	2.05	3.68	Low face, benches, tripod, very hard rock, hole spacing close, electricity, air.
5	Rhyolite conglomerate	do	.2509	.64	3.73	2.53	Low face, benches, medium hard rock, hole spacing close, steam.
11	do	do	.2667	.62	2.49	2.33	Do.
4	Granite	do	.2685	1.57	1.96	5.82	Low face, benches, tripod, hard rock, hole spacing close, steam.
9	Diorite (trap)	do	.2692	.54	3.05	2.36	Low face, benches, tripod, very hard rock, hole spacing close, steam.
13	Andesite (trap)	do	.4060	1.35	1.53	3.32	Low face, benches, tripods, very hard rock, some snake holes, spacing close, steam and electricity.
16	Rhyolite (trap)	do	.4358	2.18	.81	4.99	Low face, benches, tripods, very hard rock, spacing close, steam and electricity, steam and air.
2	Granite	do	.4482	.60	2.88	1.34	Low face, benches, tripods, very hard rock, spacing very close, steam and electricity, air.

[1] See Table 3 for spacing of holes. [2] Snake holing with tripods also at this quarry cost $1.06 per foot, and averaged 1.34 feet per hour per drill. [3] Undetermined.

should be useful in estimating on rock drilling. Table 2 gives data on the rate of drilling and the percentage of wear of the rock for the different kinds of drills.

TABLE 2.—*Relation between average feet drilled per hour by different types of drills and percentage of wear of rock as determined by laboratory test*

Tripod drills (vertical holes)			Tripod drills (snake holes)			Well drills		
Quarry key No.	Average feet drilled per hour	Per cent of wear	Quarry key No.	Average feet drilled per hour	Per cent of wear	Quarry key No.	Average feet drilled per hour	Per cent of wear
6	4.39	5.0	19	1.68	3.0	20	2.48	6.2
15	4.13	3.1	17	1.34	2.7	21	3.10	5.4
9	3.05	2.9	18	1.81	2.0	8	1.29	3.1
2	2.88	2.7				17	.65	2.7
12	2.34	3.0				7	.80	2.6
16	1.53	2.0						

FIG. 2.—BREAKING BOWLDERS WITH A STEEL BALL

At the time the studies were made little or no use was made of hammer drills in face work, although they were used in plugging bowlders. New types of hammer drills are now on the market and are widely used for bench work. They require only one man for operation and drill at a much faster rate than tripod drills. They are most successful when operated by air.

Blasting face.—Detailed data on blasting operations are shown in Table 3 in which the quarries are arranged in the order of the cost of blasting the face per ton. The highest costs were in connection with bench work in hard rock with few joints or with high faces in much jointed rock where snake holes were used. Some high costs resulted from wide spacing of vertical well-drill holes in high faces. Low costs would be expected in the large quarries but occasionally a small quarry is found in the low-cost group and a large quarry in the high-cost group.

The kind of rock, differences in joints (see Fig. 1), hole spacing and height of face were important factors affecting costs, but poor blasting often resulted in the need of further blasting for the removal of spurs and toes. Much blasting in small quarries was found to be haphazard or experimental.

Analysis showed that the tonnage brought down per pound of dynamite is but slightly related to number and spacing of drill holes. In this respect there is a great difference between the operations of large and small producers. In the item of tonnage brought down per foot drilled, which is of much greater importance, the results were much more satisfactory in the quarries classified as large and very large from the production standpoint than in those classed as medium, small, and very small. There were conspicuous exceptions to this, however.

It is concluded that there is a decided advantage in using deep, well-drill holes, widely spaced or, if the face is high and much jointed, snake holes should be used.

TABLE 3.—*Blasting face data, and direct costs arranged in order, from lowest to highest*

Quarry key No.	Direct cost of blasting face per ton of broken stone	Spacing of holes	Height of face blasted	Rock down per pound of dynamite	Rock down per foot drilled	Kind of rock and conditions
				Tons	*Tons*	
18	$0.0227	Snake holes 28 feet in	150 feet	12.9	69.09	Diabase, hard, very greatly jointed.
20	.0412	15 feet apart, 20 feet back	42 feet	4.9	24.53	Limestone, moderately jointed.
8	.0425	12 feet apart, 20 to 25 feet back	60 to 90 feet	5.8	27.24	Hard dolomite, strata slightly jointed, high face.
14	.0440	6 feet apart, 8 feet back	Benches, 18 to 20 feet	10.5	4.00	Rhyolite breccia, well jointed.
19	.0440	Snake holes 30 feet in	150 feet	6.6	32.25	Diabase, hard, highly jointed.
17	.0441	24 to 30 feet apart, 30 to 35 feet back	20 to 30 feet	6.7	[1] 53.47	Diabase, hard, many joints.
21	.0527	12 to 15 feet apart, 20 feet back	20 to 35 feet	3.9	19.23	Limestone, moderately jointed.
4	.0560	5 to 7 feet apart, 6 to 8 feet back	Benches, 14 to 18 feet	8.1	5.82	Granite moderately jointed.
12	.0574	3 to 4 feet apart, 18 feet back	Benches, 20 feet	8.0	5.19	Metamorphosed rhyolite, hard, many joints.
11	.0595	4 feet apart, 8 feet back	Benches, 14 feet	9.7	2.33	Hard conglomerate, well jointed.
5	.0609	5 to 6 feet apart, 15 feet back	Benches, 10 to 18 feet	7.3	2.53	Hard conglomerate, many joints.
7	.0616	12 feet apart, 20 to 25 feet back	70 to 90 feet	3.9	25.15	Hard dolomitic limestone, well jointed.
6	.0619	5 to 6 feet apart, 15 feet back	6 to 18 feet	6.2	2.45	Hard conglomerate, numerous joint planes.
16	.0630	4 feet apart, 6 to 8 feet back	Benches, 15 feet	5.4	4.99	Altered rhyolite, greatly jointed.
1	.0690	5 feet apart, 6 to 8 feet back	Benches, 20 feet	7.0	3.68	Hard granite and porphry, few joint planes.
3	.0794	10 feet apart, 20 feet back	60 feet	5.5	18.70	Granite, moderately jointed.
22	.0800	Snake holes 16 to 22 feet in	75 to 150 feet	6.0	8.51	Marble, hard, fairly jointed.
15	.0886	4 feet apart, 8 feet back	16 feet	5.1	2.31	Trachytic rhyolite, numerous incipient joints.
9	.0913	5 feet apart, 8 feet back	Benches, 20 feet	5.5	2.36	Hard and tough diorite, tight and few joints.
23	.0913	24 feet apart, 20 feet back	75 feet	2.2	24.09	Slate, very greatly jointed.
13	.0968	3 to 4 feet apart, 18 feet back	Benches, 18 to 20 feet	4.1	3.32	Altered andesite, hard, joints few and tight.
2	.1016	3 to 5 feet apart, 2 to 10 feet back	Benches, 20 feet	3.4	1.34	Hard granite, and porphry, few joint planes.
10	.1389	6 to 8 feet apart, 8 feet back	do	3.0	6.48	Diorite, hard and tough, relatively few joints.

[1] Includes well-drill holes and snake holes.

TABLE 4.—*Showing bowlder-breaking methods and costs per ton of broken stone produced*

Quarry key No.	Initial crusher size	Methods of breaking	Direct cost of drilling bowlders	Direct cost of blasting bowlders, mud capping, and block holing	Direct cost of steel balling bowlders	Direct cost of sledging bowlders	Direct cost of breaking bowlders, all methods	Average daily production in tons	Remarks
1	No. 5 gyratory	Block holing and sledging	$0.0714	$0.0827	None.	$0.3727	$0.5268	172	Granite, many bowlders, few fracture planes.
2	Two No. 5 gyratory independent units.	do	.1204	.0392	None.	.3149	.4745	260	Same quarry as key No. 1, following year, fewer bowlders due to change of method of blasting.
3	Jaw opening, 10 by 20 inches.	Steel balling and sledging	None.	None.	$0.0513	.0691	.1204	176	Well managed.
4	do	Mostly steel balling and sledging.	.0330	.0037	.0687	.1374	.2428	125	Same quarry as key No. 3, following year. Owner-manager sick, wages higher, labor much less efficient.
5	Two jaw crushers, 10 by 20 inches.	do	.0010	.0003	.2394	.2506	.4913	73	Very poor management and supervision.
6	No. 6 gyratory	do	.0003	.0002	.0175	.1517	.1697	292	Same as key No. 5, following year, good supervision.
7	Five No. 6 and one No. 4 gyratories.	Mostly mud capping; some block holing.	.0099	.0770	None.	None.	.0869	1,454	Large-scale operation, steam shovel to handle rock, large initial crushers.
8	Five No. 6 gyratories	Block holing and mud capping.	.0245	.1272	None.	None.	.1517	770	Do.
9	Jaw opening, 36 by 42 inches.	All block holing	.0514	.0184	None.	None.	.0698	234	Rock broke well on blasting face, fewer bowlders, large initial crusher, steam shovel.
10	No. 8 gyratory	Block holing and sledging	.0430	.0261	None.	.1627	.2318	532	Bowlders very tough but labor wasted in operation of sledging.
11	Jaw opening, 10 by 20 inches.	Steel balling, block holing	.0216	.0035	.1392	.2427	.4070	66	Inefficiency conspicuous, cost of steel balling and sledging much too high.
12	No. 6 gyratory	Steel balling and sledging	None.	.0009	.0487	.0939	.1435	214	Bowlders very tough.
13	Jaw opening, 16½ by 29 inches.	Block holing and sledging	.0284	.0201	None.	.0847	.1328	170	Numerous incipient fractures in bowlders, hence sledging cost low.
14	Jaw opening, 14 by 21 inches.	Mostly steel balling and sledging; some block holing.	.0196	.0024	.0945	.1804	.2969	82	Inefficiency, costs of steel balling and sledging too high.
15	do	Sledging	None.	None.	None.	.2501	.2501	86	No bowlder blasting permitted (city ordinance), hence every large bowlders broken by sledging, and cost of that item high.
16	Jaw opening, 36 by 18 inches.	Sledging and steel balling	None.	None.	.0073	.2063	.2136	180	Small bowlders hard to break, hence sledging cost high.
17	Jaw opening, 48 by 72 inches.	A little mud capping	None.	.0097	None.	None.	.0097	1,820	Large-scale plant, large initial crusher, steam shovel for loading, hence low costs.
18	No. 7½ gyratory	Very little bowlder breaking; mud capping.	.0047	.0113	None.	.0006	.0166	264	Rock broke well on blasting, relatively few bowlders, relatively large initial crusher, steam shovel for loading.
19	Jaw opening, 48 by 59 inches.	do	.0029	.0108	None.	None.	.0137	600	Do.
20	Jaw opening, 48 by 60 inches.	A little mud capping and block holing.	.0084	.0273	None.	None.	.0357	1,977	Conditions similar to those at key No. 17, see above.
21	Nos. 10, 9, and 8 gyratories.	do	.0093	.0171	None.	None.	.0264	1,772	Do.
22	No. 6 gyratory	Block holing and mud capping.	.0565	.1140	None.	.1351	.3056	176	Too many bowlders, due to blasting method.
23	No. 7½ gyratory	None	None.	None.	None.	None.	None.	94	Many joint planes, rock finely broken on blasting, hence no bowlders.

TABLE 5.—*Loading and delivery data*

Quarry key No.	Type of quarry	Average daily production in tons	Cost of loading per ton	Tons loaded per labor hour	Loading, common labor, rate per hour	Method of loading	Cost of delivery per ton	Cost per day, horse, cart, and driver
1	Open face	172	$0.1866	2.5	$0.45	By hand into high-sided tramcars	$0.0862	
2	do	260	.2056	2.5	.50	do	.1758	
3	do	176	.1235	4.6	.50	By hand into low-scale boxes resting on ground	.1283	
4	do	125	.0797	6.0	.50	do	.1347	
5	do	73	.2010	2.67	.50	By hand into high dump carts	.2071	$5.50
6	do	292	.1284	[1] 4.0	.50	By hand into low-scale boxes resting on ground	.0740	
7	Open face and pit	1,454	.1041	3.4	[2] .30	By steam shovel mostly, part by contract, hand labor	.0505	
8	Open face	770	.1667	3.2	[2] .30	By hand into tramcars, by contract, hand labor	.0667	
9	do	234	.1409		.39	Steam shovel to tramcar	.0328	
10	do	532	.0976		.44	Steam shovel and hand labor into low tramcars	.0382	
11	Pit	66	.2443	2.27	.50	Mostly by hand into high dump carts	.1251	[3] 4.50
12	Open face	214	.1521	3.05	.44	do	.1186	5.50
13	do	170	.2202	2.29	.45	do	.1301	5.00
14	do	82	.2040	2.7	.50	do	.1445	5.25
15	do	86	.3280	1.4	.42	do	.1784	5.40
16	Pit	180	.1788	[4] 2.9	.50	By hand into low-scale boxes resting on ground	.1459	
17	Open face	1,820	.0535		.40	Steam shovel	.0664	
18	do	264	.0902		.43	do	.0229	
19	do	600	.0729		.40	Electric shovel	.0376	
20	Pit	1,977	.0462		.50	Steam shovel	.0838	
21	Open face	1,772	.0538		.50	do	.0627	
22	Pit	176	.1294	3.5	[2] .50	By hand into tramcars, by contract, hand labor	.1514	
23	Open face	94	.1900	3.2	[2] .60	By hand into dump carts, contract labor	.0695	[5] 2.00

[1] Rate of tons per hour would have been still higher if more of the material loaded had not been unnecessarily handled twice.
[2] Calculated from contract basis.
[3] One driver for two 1-horse carts.
[4] Men lost time waiting for delivery to crusher.
[5] Cost of care and feed only; no driver employed.

Quarry key No.	Method of delivery	Average length of haul and grade	Power: Kind	Power: Cost	Power: Unit	Remarks
1	Cars pushed and switched by men, then hoisted up incline.	400 feet down grade for car pusher. Incline by hoist engine.	Coal	$10.00	Ton	Complicated method of delivery.
2	Two independent units to two crushers. Cars pushed and switched by men, then hoisted up incline.	do	do	10.75	do	Capacity of plant doubled by adding separate and independent unit, production actually increased only 50 per cent. Method of delivery complicated.
3	Derrick swing to pier; thence by car pusher down grade to crusher.	375 feet down grade	do	10.00	do	Loading and delivery methods excellent.
4	do	do	do	8.50	do	Do.
5	1-horse dump carts	450 feet upgrade	Men and horses			These methods always expensive.
6	By derrick direct to crusher.	150 feet	Coal	9.50	Ton	Excellent system, cost would have been much lower but for unnecessary rehandling much of material during loading.
7	Steam locomotive mostly, part by mule to incline, then steam hoist.	800 feet, about level, small per cent up incline.	do	5.00	do	Typical of large-scale operations, these costs somewhat raised because of taking part of stone from a pit.
8	Men and mules on tram track.	500 feet level	Men and horses			Typical of large operations.
9	Cable pull by hoisting engine, automatic dump.	300 feet up gentle grade	Coal	10.00	Ton	One of the most efficient delivery systems, large, low tramcars, gentle upgrade, steam shovel expense too high, gravity return.
10	Horses and men	350 feet down grade	do	9.00	do	Good methods, well managed.
11	1-horse dump carts, also derrick hoist.	300 feet steep upgrade	do	10.00	do	Methods expensive, typical of most small-scale plants.
12	1-horse dump carts	200 feet upgrade	Men and horses			Do.
13	do	400 feet upgrade	do			Do.
14	do	225 feet upgrade	do			Do.
15	do	150 feet slight upgrade	do			Methods expensive, typical of most small-scale plants, men wasted time excessively.
16	By derrick and overhead cable system.	500 feet up	Coal	10.75	Ton	Costs could have been lower but for troubles with overhead cable system.
17	Steam locomotives	3,000 feet down grade	do	5.54	do	Typical of large-scale operations.
18	Car pusher, gravity	200 feet down grade	do	7.75	do	Power loading, gravity delivery, short distance and good management led to lower costs.
19	Gasoline locomotive	300 feet down grade	Gasoline Electricity	.24 .017	Gal Kwh	Typical of large-scale operations.
20	Steam locomotive to pit crusher, thence by hoist.	2,000 feet crusher in pit, then conveyor to main plant.	Coal	7.50	Ton	Do.
21	Steam locomotive	2,300 feet level	do	10.65	do	Do.
22	First by horse to foot of incline, second by electric hoist to crusher.	250 feet up by hoist engine.	Electricity	0.17	Kwh	Complicated method of delivery due to pit-type quarry.
23	First, by 1-horse carts to incline; second, by steam hoist to crusher.	125 feet level, then hoist engine to crusher in top of plant.	Coal	7.00	Ton	Loading done by contract labor at relatively higher price. Exceptional low cost for horses kept delivery costs low. Haul in quarry 125 feet, thence by bucket hoist 42 feet.

The 10 quarries with the lowest combined cost of drilling and blasting face used one of these two methods. Cost of the two items is not necessarily affected by production volume as the quarries ranking highest and lowest in this cost had about the same output.

STEEL BALLING THE MOST SATISFACTORY METHOD OF BOWLDER BREAKING

Bowlder breaking.—So many factors enter into bowlder-breaking costs that generalizations are difficult to make and the conditions at each quarry, as given in Table 4, must speak for themselves. The factors affecting the cost are efficiency of management, size of crusher opening, method of loading, drill-hole spacing and blasting, and the character of the rock itself. Illustrating the wide range in conditions it will be noted that at quarry No. 23 there was no bowlder breaking, while at No. 1 the cost amounted to more than 52 cents per ton of broken stone produced.

While it is difficult to give general conclusions, the studies do indicate that the breaking of bowlders by the steel-ball method can be used in many quarries to advantage. With this method (see fig. 2) one or more derricks operated by three-way hoisting engines are placed at a convenient distance from the face and by means of a chain and cable bowlders too large for sledging are dragged from the face and arranged beneath the derrick. The derrick is used to lift a steel ball weighing about 2 tons to a height of from 40 to 70 feet and drop it on the bowlder.

Great variation was observed in the success of this operation, but where skillfully directed, it was considered very successful. Comparing the cost of bowlder breaking at two small-scale quarries, where steel-balling was not used, with three as nearly similar quarries as it was possible to find where steel-balling as best practiced was used, it was found for the former that the cost of breaking bowlders per ton of broken stone averaged 26 cents while at the latter it averaged 14 cents.

Fig. 3.—Sledging Is a Large Item of Expense at Most Small Quarries Where Rock Is Reduced by Hand Labor for Small Initial Crusher

At medium and small-scale quarries the derricks used for steel-balling may also be used for transporting stone to the crusher and thus lower the cost of this item. If the distance is small, low scale boxes may be loaded and swung directly to the crusher, while if it is great they may be used for dumping into tram cars.

Pumping and disposal of waste.—These operations were so unusual that they did not enter materially into the production cost with one exception. In the pit-type quarries open seams permitted the escape of water except at one quarry where pumping cost \$0.0256 per ton of broken stone. At only two quarries was it necessary to remove waste from the quarry floor. At one of these the cost was a fraction of a cent per ton of broken stone while at the other it amounted to 12 cents. At the latter quarry stripping often slid to the quarry floor and clay seams occurred between the beds of marble.

DUMP CART DELIVERY EXPENSIVE

Loading and delivery.—Loading and delivery have been considered together because a loading method must take into consideration the method of delivery.

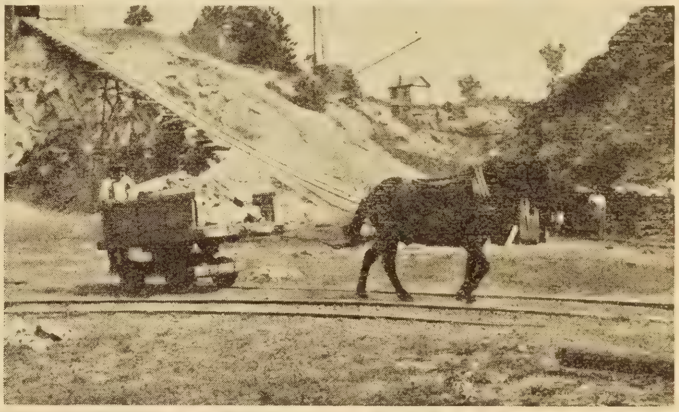

Fig. 4.—The Upper Picture Shows an Expensive Delivery Method, Which Cost 12½ Cents Per Ton. The Middle Picture Shows a Combination Method, Horses Drawing 2½-Ton Cars to the Incline and Thence by Steam Hoist. The Bottom Picture Shows an Economical Method, Coasting Cars Down Incline at a Cost of 3¾ Cents per Ton

Methods of delivery varied widely (see fig. 4) as would be expected in a series of plants selected because of varying conditions. Among the simplest methods were direct swing and hoist derricks, one-horse dump carts, tram cars pushed or coasted down grade to the crusher, direct cable haul to the crusher, and gasoline and steam locomotives. Numerous combinations of methods were found, most of which resulted from the problems of pit-type quarries. Table 5 shows the methods and costs of these operations at the various quarries.

In the item of loading it appears that many quarries could effect a considerable saving by a change in methods. It was found that a low receptacle could be loaded about twice as fast as a high one. (See fig. 5.) Five quarries using high dump carts loaded at a rate of 2.6 tons per hour per man, two using high tram cars loaded at a rate of 2.5 tons per hour, while two using low scale boxes loaded at a rate of 5 tons per hour. Assuming the same wage rate at all of the quarries the respective costs per ton were 21 cents, 20 cents, and 10 cents. Loading at five quarries using power shovels averaged 6 cents per ton, the range being from 4 to 9 cents.

Loading by contract labor was done at four quarries and was much more efficient than day labor. The contract labor averaged 3.3 tons per hour while the average for eight quarries using day labor was 2.6 tons per hour. In both groups the material was loaded into high tram cars and dump carts.

There was found to be a wide range in the cost of delivery, which demonstrates that some of the quarries were using poor methods, although some of the high rates could be attributed to adverse conditions.

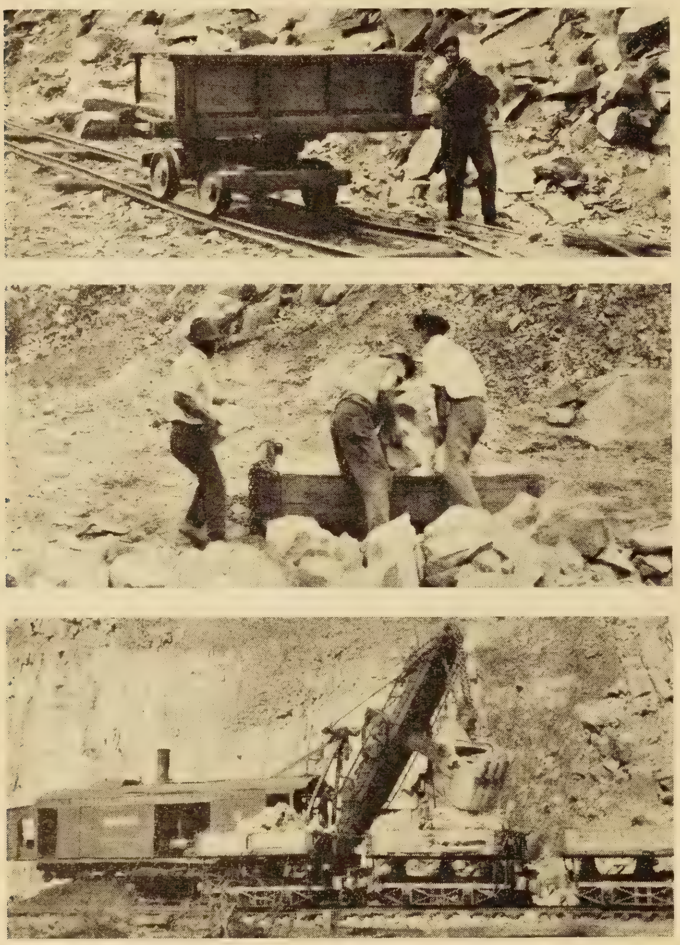

Fig. 5.—Height of Cart or Car Is an Important Factor in Loading in the Quarry Shown in the Upper Picture. Loading Was at a Rate of 2.4 Tons Per Hour, While in the Middle Picture the Rate Was 6 Tons Per Hour. The Bottom Picture Shows Large-Scale Steam-Shovel Loading

The lowest cost of delivery, $0.0229 per ton, resulted from coasting tramcars of 6-ton capacity an average distance of 200 feet down a gentle grade and returning the cars to the face by two men. The second lowest cost of $0.0328 per ton was incurred at a quarry where tramcars of 6-ton capacity were hauled to the crusher by a hoisting engine operating a cable a distance of 300 feet up a gentle grade which permitted a gravity return. Dumping was automatic. The third lowest cost resulted from gasoline-locomotive delivery of tramcars of 2-ton capacity a distance of 300 feet down a gentle grade. The average cost of delivery at six quarries by one-horse dump carts an average distance of 262 feet and in general slightly up grade was $0.1506, or about five times the average for the three quarries mentioned.

The average cost of delivery by steam locomotives over distances ranging from 2,300 to 3,000 feet was $0.0645 per ton. At pit-type quarries where steam locomotive hauls of 800 to 2,000 feet were combined with incline hoists the average cost per ton was $0.0672.

The use of the derricks used in steel balling for lifting and swinging scale boxes loaded with stone to the crusher platform has already been mentioned. At the quarry where this arrangement was used it resulted in a cost of $0.074 per ton and the cost of loading into the low scale boxes was also low. Figure 6 shows different methods of dumping at the crusher.

Crushing and screening.—Data on the cost of crushing and screening is presented in Table 6. Of the two quarries showing lowest costs, quarry No. 6 had an average daily production of only 292 tons of conglomerate rock which was easy to break, and quarry No. 21, 1,772 tons of limestone. Both used electric power. At quarry No. 6 a No. 6 gyratory crusher was used and very little crusher feeding was necessary.

Table 6.—*Crushing and screening data, and direct costs arranged in order, from lowest to highest*

Quarry key No.	Cost per ton of broken stone	Power elements		Average daily production in tons during period
		Kind	Cost per unit	
6	$0.0493	Electricity	$0.027 per kwh	292
21	.0630	do	$0.018 per kwh	1,772
19	.0724	do	$0.017 per kwh	600
18	.0764	do	$0.030 per kwh	264
16	.0816	Coal	$10.75 per ton	180
10	.0823	Electricity	$0.027 per kwh	532
17	.0827	do	$0.015 per kwh	1,820
7	.0836	do	$0.009 per kwh	1,454
2	.0896	Coal Electricity	$10.75 per ton $0.027 per kwh	260
15	.0904	Electricity	$0.030 per kwh	86
20	.1013	Coal	$7.50 per ton	1,977
13	.1060	Electricity	$0.027 per kwh	170
8	.1069	do	$0.009 per kwh	770
12	.1103	do	$0.027 per kwh	214
4	.1170	do	$0.027 per kwh	125
22	.1215	do	$0.017 per kwh	176
3	.1271	do	$0.027 per kwh	176
14	.1408	do	$0.027 per kwh	82
11	.1437	do	$0.027 per kwh	66
23	.1569	Coal	$7 per ton	94
9	.1737	do	$10 per ton	234
1	.2052	do	$10 per ton	172
5	.3423	do	$11.75 per ton	73

Two of the quarries listed were under the same management and crushing the same kind of rock with modern plants. At one of these quarries steam was used for power, at the other electricity. Crushing with steam power cost $0.1013 per ton while crushing with electric power cost only $0.0630 per ton. This difference in cost is believed to be largely the difference in cost of power.

GENERAL CONCLUSIONS

Table 7 shows the total direct cost per ton for each of the quarries with information as to the kind and size of quarry. Comparisons of all the quarries as to cost of the various operations show that none of them maintained a uniform high position on all items. There is a general tendency for a quarry to be among the highest group with regard to one operation and much lower down the scale with regard to the next. The total direct costs do in general vary inversely with production, but the difference would be less conspicuous if these costs included rental, or interest and depreciation on power equipment. It is believed that a relatively small-scale quarry can be operated with direct costs lower than the average, or even the best record of the large-scale quarries if good management is combined with a good layout, good equipment, electric power, and favorable conditions of face and rock. Results at quarry No. 18 demonstrate this, as it had the lowest direct production cost of any of the 23 quarries and yet had an average daily production of but 264 tons.

TABLE 7.—*Total direct production costs, kind of rock, magnitude of operation, and price of labor per hour*

Quarry key No.	Direct production cost per ton	Kind of rock	Magnitude of operation	Price of labor per hour
1	$1.3179	Granite	Small	$0.45
2	1.5250	do	Medium	.50
3	.6606	Altered granite	Small	.50
4	.8987	do	do	.50
5	1.5648	Conglomerate	Very small	.50
6	.7328	do	Medium	.50
7	.4242	Siliceous dolomite	Very large	.30
8	.5595	Dolomite	Large	.30
9	.7931	Altered diorite	Medium	.39
10	.7077	Diorite	Large	.44
11	1.2503	Altered rhyolite	Very small	.50
12	.7261	do	Medium	.44
13	1.1095	Altered andesite	Small	.45
14	1.0684	Rhyolite breccia	Very small	.50
15	1,1547	Trachytic rhyolite	do	.42
16	1.1187	Altered rhyolite	Medium	.50
17	.2828	Altered diabase	Very large	.40
18	.2535	Diabase	Medium	.43
19	.2962	Altered diabase	Large	.40
20	.3471	Dolomite limestone	Very large	.50
21	.3130	Limestone	do	.50
22	.9937	Marble	Small	.50
23	.6546	Siliceous slate	Very small	.39

At many plants, especially the smaller ones, only a crude system of bookkeeping is used. As a rule the distinction is not made between bookkeeping and cost keeping. If total costs are too high, the owner does not know where the trouble lies. Indeed, within a wide range of costs that lie inside the profit line, he is likely not to know whether any or all of his costs are too high or what methods, if any, should be changed, or whether a foreman or superintendent is getting the best possible results from labor. As this is true with direct costs so it is true with indirect costs, and a very common fault is to take little or no account of such important factors as depreciation and depletion.

The writer studied costs at three quarries located in the suburbs of a large city which were owned and operated by a contractor doing a large business. Most of the broken stone produced at these quarries was used on his contract jobs. Such stone as was sold was sold by the yard and measured in the delivery trucks. All the rest of his product was sent by truck to centrally located scales for weighing. Incredible as it may seem no record was made as to which of the three quarries the stone came from and no individual records were kept of their output. The owner only knew the total production of the three quarries which were several miles apart and under separate managements. As a matter of fact one of them was very badly managed and direct costs were about as high as they could be made.

With reference to face breaking, there is no question but that modern types of hammer drills should be used for bench work at least, and that in the long run the use of air instead of steam is economical in spite of the necessary investment in a compressor. Well drill ng and wide spacing of holes and snake holing high faces reduce very largely the total costs of face breaking, and the type of explosive used should be determined by both experiment and advice from manufacturers.

Where the scale of operations is not too large the most satisfactory method of bowlder breaking is steel balling, although this might not be applicable to the breaking of bowlders in some of the tough diabases which are sometimes quarried. This must be determined experimentally. Comparative results in these studies indicate that mud capping is the most expensive method.

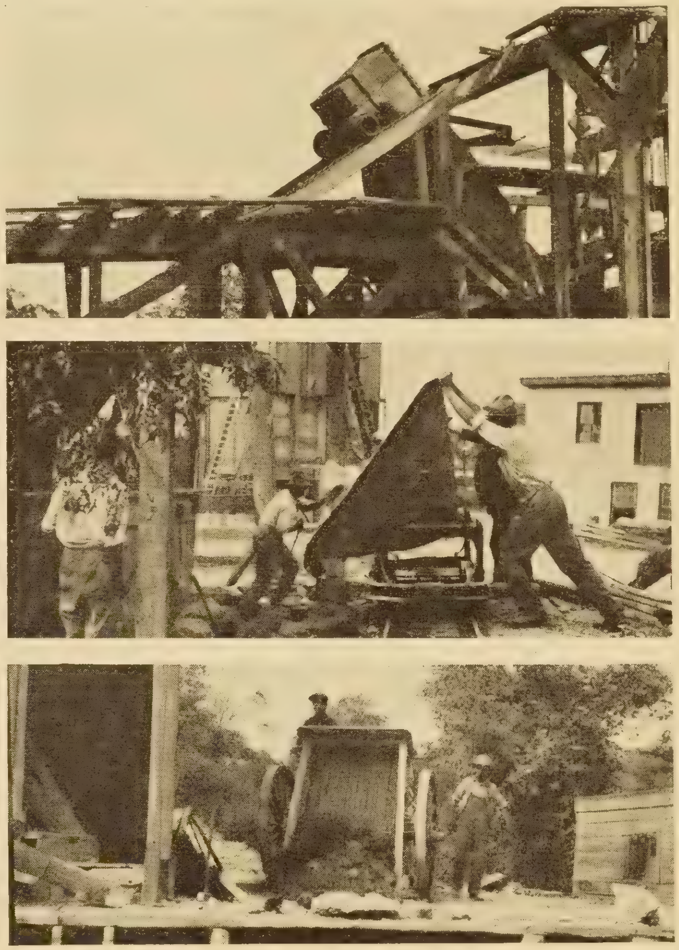

FIG. 6.—THE TOP PICTURE SHOWS AN ARRANGEMENT FOR AUTOMATIC DUMPING OF 5-TON TRAM CARS, THE MIDDLE PICTURE SHOWS A GOOD HAND-DUMP TRAM CAR, AND THE LOWER PICTURE SHOWS A CART DUMPING ON CRUSHER PLATFORM, REQUIRING ABOUT HALF OF STONE TO BE HAND FED

Loading by hand into high receptacles costs from two to three times as much as loading into low-scale boxes. Steam shovels may be economically operated in comparatively small quarries and a great deal of sledging expense saved if a reasonably large initial crusher is used.

The horse-drawn dump cart is most expensive as to both loading and delivery. Delivery by derrick, directly, and by tramcars into which the contents of derrick-handled scale boxes are dumped, are relatively inexpensive, and the results as shown in costs of delivery at quarries Nos. 3, 4, and 6 do not do justice to the method, because in each case with insignificant ncrease of expense the number of tons delivered could have been more than doubled, while the results by the dump-cart method were for the maximum capacity of operations, and any increase in capacity called for a proportional increase in expense.

For distances up to 300 feet at least delivery is very economically accomplished, if grade permits it, by hand-pushed or horse-drawn tramcars of 2 to 4 tons capacity. Of all the methods of delivery studied the cheapest was the method using automatic dumping, 6-ton tramcars (steam-shovel loaded) pulled up a gentle grade to the crusher by a drum hoist and returned to the quarry face by gravity. A few posts set in the quarry floor as cable guides made it possible to draw these tramcars around curves, and thus made all parts of the face available for this method without unnecessary track complications or extra assistance other than such as could be given as needed by the quarry foreman. Power locomotives are of course necessary for long hauls and large operations, and simple arrangements cut down delivery expense.

Crushers above quarry floors and complicated track and switch arrangements result in high cost of delivery to the crusher. Pit-type quarries, as would be expected, have relatively high costs for delivery.

The kind of power used has much to do with costs. The more extensively electricity is used the more economically are most of the operations conducted.

At most small-scale quarries the initial crusher is too small. The saving through use of a small crusher is usually lost several times over by increased sledging costs.

For large-scale operations plants are usually designed, methods adopted, and layouts planned by competent engineers. On the other hand, for small plants these features usually are the result of expediency, experiment, or guesswork. Sometimes they result from previous experiences, good or bad, where very different conditions prevailed. As a rule insufficient attention is paid to the relations between the cost of bowlder breaking and the methods of loading, drilling, blasting, and the size of initial crusher. The method of one operation is dependent for success upon that of another. In other words, operation methods are closely interdependent.

Whenever possible i1 establishing a broken-stone plant the probable duration and both present and probable future magnitude of operations should be first determined. A large plant is, of course, not justified by small ledge holdings, and the relation between these two must be recognized. Topography must be studied and, if possible, a site for the plant procured below the level selected for the quarry floor and as near the face as blasting operations will safely permit. The ledge itself should be studied with reference to joint planes and seams, and the rock itself tested for percentage of wear. Conditions as to the former will indicate drill-hole spacing and blasting methods likely to be most satisfactory, and the latter will give information as to the probable cost of drilling.

If these considerations, together with the foregoing notes on various operations, are kept in mind they should lead to production costs lower than those now incurred at very many existing plants.

Finally it should be stated that no plant can afford to omit the keeping of costs on each unit of operation.

(Continued from p. 162)

PRELIMINARY AND FINAL SOIL CLASSIFICATION

Suppose we get a shipment of soil samples from a certain section of the country and we face the problem of comparing the soils with those previously received from other districts. Since an exhaustive investigation of every one of the samples would be financially forbidding, we have to start with a rapid survey of the material for the purpose of separating the soils into groups with fairly similar properties. This survey consists in determining for each soil the limits and in representing the limits in a regional soil diagram of the type of Figures 8 to 12 and Figure 14. The next step consists in classifying the soils according to their lower liquid limit.

For material with a low liquid limit, a difference of 5 per cent between the liquid limits has obviously more weight than the same difference for materials with a very high liquid limit. Hence the classification according to the liquid limit is made as follows:

Class	Liquid limit per cent
0	10 to $10\times 2^{1/2} = 10$ to 14.2
I	$10\times 2^{1/2}$ to $10\times 2 = 14.2$ to 20.0.
II	10×2 to $10\times 2^{3/2} = 20.0$ to 28.4.
III	$10\times 2^{3/2}$ to $10\times 2^{2} = 28.4$ to 40.0.
IV	10×2^{2} to $10\times 2^{5/2} = 40.0$ to 56.8.
V	$10\times 2^{5/2}$ to $10\times 2^{3} = 56.8$ to 80.0.
VI	10×2^{3} to $10\times 2^{7/2} = 80.0$ to 113.6.

If we have found, for instance, that a soil belongs in the class V, indicating a very high liquid limit, we know that the soil may either be excessively fine grained or else very rich in flexible, scale-like particles. The soil will be very different according to whether the first or the second assumption is correct. Hence, within each one of these groups distinction is made between soils with a low, medium or high plasticity index. The occurrence of a high liquid limit with a low plasticity index is very probably due to excessive fineness of the soil, while another soil with a high index is very probably distinguished by an abundance of scale-like particles which in turn seems to be associated with a very marked tendency to swell in contact with water. The shrinkage limit serves as an additional check on the degree of relationship between the members of each group.

However, a final classification should not be based on data as vague as the conclusions we can draw from the limits. Hence, in order to establish a permanent foundation for soil classification, we have to go one step further and investigate what the moduli of compression, of expansion, and the coefficients of permeability of the members of the different groups are. Since these moduli depend on precisely the same factors as the limits, there must be a numerical relationship between them and the knowledge of this relationship will ultimately bring the interpretation of the limits to a certain state of perfection. However, the classification itself will have to be based on the numerical values of the three characteristic constants, viz, the modulus of compression, the modulus of expansion, and the coefficient of permeability at a standard pressure, because these are the only numerical values which have a direct bearing on the mechanical properties of the subgrade, including its resistance.

Any well organized soils laboratory will ultimately arrive at empirical rules for translating the results of its own limit determinations into physical terms. But, on the other hand, it is very doubtful whether the same rules will ever apply to limits determined by different laboratories. In spite of carefully standardized instructions, a certain personal element inevitably adheres to every limit determination and, for this reason, the necessity for checking the limits against the results of well defined physical tests will always exist.

CONCRETE COMPARED WITH TIMBER FOR HIGHWAY BRIDGE FLOORS

Reported by O. L. GROVER, Bridge Engineer, Bureau of Public Roads

THE RENEWAL of plank floors on bridges is a large item in the annual expense of bridge maintenance; and aside from its expense, which is heavy, wooden flooring is noisy and causes excessive vibration of the structure and objectionable jolting of traffic when improperly laid or in a worn condition. While treated timber is expected to resist decay for about 25 years, unprotected flooring has a service life of from 3 to 6 years only, depending on the amount and character of traffic; and on bridges which carry a large volume of traffic or heavy units, such a floor wears out so fast that maintenance is difficult and the traffic is more or less unsatisfactorily served at all times in spite of heavy outlays for repairs.

Figures obtained by averaging the cost of maintaining unprotected timber floors on several steel bridges over a number of years show what the cost of upkeep may amount to on bridges carrying main-route traffic. In the District of Columbia for a 30-year period prior to 1926 the average cost for repair and renewal was about 3½ cents per square foot per annum, or about 70 cents per lineal foot for a 20-foot roadway, while for a 10-year period prior to 1926 the average was higher chiefly because of changes in the amount and character of traffic. In general it may be said that the experience of the past 10 years indicates a cost of about 6 cents per square foot per annum, or $1.20 per lineal foot for maintaining a 20-foot floor on bridges carrying present traffic on through highways near cities.

Such costs are excessive. In the effort to reduce them and improve the service of the unprotected floor, where the strength of the bridge has been adequate to carry the added dead load, bituminous surfacing has been tried as one method. Another which has met with some favor is the addition of steel-plate traffic treads; a third is the addition of a longitudinal or diagonal layer of timber as a wearing surface; and the use of planed flooring instead of rough plank has also been given a reasonable trial. The object of the latter method is to reduce wear by building a smooth floor, and this is accomplished by the uniform thickness of the planed flooring. The beneficial effect of this method, however, is gradually lost by reason of the wear and replacement of occasional pieces with new pieces, the depth of which naturally varies considerably from that of the adjacent flooring.

For a 20-foot roadway the average unit prices of these are approximately as follows:

Bituminous surfacing 2 inches thick, per lineal foot	$5. 55
Four lines of steel tread, 24 inches wide by $\frac{5}{16}$ inch thick, per lineal foot	8. 96
Four lines of steel tread, 24 inches wide by $\frac{3}{16}$ inch thick, per lineal foot	5. 60

It thus appears that the 2-inch bituminous surfacing costs $3.41 less per lineal foot than the 5/16-inch treads and 5 cents less than the 3/16-inch treads. But these comparisons are naturally affected by the relative prices of steel and bituminous materials, and since the former vary from about 7 to 11 cents a pound and the latter from approximately $1.75 to $3.50 a square yard the relative cost of the two methods may vary considerably from the above averages which are based on steel at 8 cents and bituminous surfacing at $2.25 a square yard. Whichever, under the particular conditions obtaining, may be the cheaper, it is probable that the reduction in the cost of maintaining the floor will be more than sufficient to pay for the covering. But the resulting construction is still open to some of the same objections which pertain to unprotected plank floors, and some of these are serious.

With these objections in mind Dr. J. A. L. Waddell declared in 1919: "Wooden floors on highway bridges are now obsolete," adding that "no modern bridge engineer should be guilty of designing or building any more of them." He gave as his reasons for these very positive statements the following three reasons:

"First. The ordinary plank floor supported on wooden joists is not strong enough to carry with safety the heavy, concentrated live loads produced by rapidly moving auto trucks.

"Second. The danger from fire is so great that it does not pay to submit the entire construction to the chance of destruction by the burning of the timber in the deck.

Fig. 1.—Steel Viaduct With Concrete Floor Carried on Concrete Pedestals. View Shows the Span Over Railroad Track Incased in Concrete

"Third. The upkeep of a wooden floor is far higher than that of a paved floor on a concrete base; and the difference in the future will steadily increase, because the cost of timber will rise with its augmenting scarcity."

COSTS OF CONCRETE AND TIMBER FLOORS COMPARED

General adoption of the concrete floor, for which Doctor Waddell so emphatically declared seven years ago, has been retarded to a certain extent by the unsatisfactory experience with the older type, used extensively in and near cities, consisting of a concrete bed for block or bituminous surfacing carried by steel buckle plates. This was the type used before the adoption of reinforced concrete slabs, and it has proved to be expensive both to construct and to maintain. The defects usually noted are broken buckle plates, especially where the buckle is turned up, broken and loose concrete, and frequent expensive repairs resulting from the breaking up of the pavement and holes in the floor.

The reinforced slab design is not open to these objections; and in common with the older type it possesses the very desirable quality of rigidity which inspires the confidence of those who ride over it. To the engineer it also suggests the idea of heavier weight, and the first thought is likely to be that extra steel will be required to carry it, so much, perhaps, that the cost of the bridge with concrete floor may exceed the cost of a timber-floored structure. So far as the trusses and floor beams are concerned this assumption is correct, but on account of the possibility of using deeper I beams for stringers and a wider spacing the saving in stringer weight may be nearly enough to compensate for the increased weight of trusses and beams.

To show the effect of using a concrete instead of a wooden floor upon the cost of actual structures of various types a number of cases are cited below in which comparative estimates or alternate bids on the two types of construction are available.

Case 1.—The structure shown in Figure 1 consists of a steel viaduct on concrete pedestals and open abutment bents with riprapped embankment around the ends of the fills. It crosses a railroad track and a creek about 70 feet wide. The entire structure is 379 feet in length, with span lengths as follows:

	Feet
1 deck truss	65
2 deck I-beam spans, each 40 feet	80
5 deck I-beam spans, each 32 feet	160
2 deck I-beam spans, each 37 feet	74

The roadway width is 20 feet.

For this structure alternate designs were made, one with a concrete, the other with a timber floor, both to carry a concentrated live load of two 15-ton trucks with 67 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact.

While the preliminary estimates indicated that the cost of the structure would probably be increased 2½ per cent by the concrete floor, the contract was let at no increase. One of the 40-foot I-beam spans over the railroad track, as constructed, has its beams encased in concrete. This was not provided for in the alternate timber-floor scheme. The estimate of the cost of the concrete-floored structure complete, based on bid prices and including 10 per cent for incidentals, is $44,000.

Case 2.—Another bridge, now under construction, crosses a river the bed of which is erodible to a considerable depth. It is designed to carry two 15-ton trucks with 67 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact.

Alternate designs for concrete and timber-floored structures were made for two arrangements of spans including varying span lengths. For one arrangement the estimate of the cost of the concrete-floored structure was about 2 per cent higher than that for the wooden-floored design; for the other arrangement the concrete floor caused an increase of about 4 per cent in cost.

The first arrangement with concrete floor and a roadway width of 18 feet is now under construction. It consists of three steel spans between piers 130, 190, and 320 feet apart, and eight timber trestle spans, each 19 feet in length, making a total length of 792 feet. The contract price for this structure, exclusive of approaches, is $180,000.

COST INCREASED ONLY 2 PER CENT BY CHANGE FROM WOOD TO CONCRETE

Case 3—In another case which recently came to the writer's attention a design was made for two spans with an 18-foot roadway, one 180 and the other 200 feet in length, with a concrete floor, after preliminary designs had been made for the same spans with a timber floor.

In each case the design was made for a concentrated live load of two 15-ton trucks with 80 per cent of their weight on the rear axle, plus an allowance of 30 per cent of the live load stress for impact.

For the entire structure the estimates showed a margin in favor of the wooden-floored design amounting to $617, the cost with concrete floor being $33,425. For the two spans the difference was thus shown to be about $1.63 per foot or less than 2 per cent of the total cost of the wooden-floored structure. For the 180-foot span the difference was $1.58 per foot; for the 200-foot span about $1.67 per foot. The comparative estimates for the two designs of each span were as follows:

Comparative estimates of cost of the 180-foot span

1. WITH CONCRETE FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Item	Quantity	Unit price	Total
Concrete	88 cubic yards	$22.00	$1,936.00
Structural steel	214,000 pounds	.06	12,840.00
Reinforcing steel	12,500 pounds	.055	687.50
Total			15,463.50

2. WITH CREOSOTED TIMBER FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Item	Quantity	Unit price	Total
Structural steel	210,000 pounds	$0.06	$12,600.00
Creosoted timber	25,800 feet b. m	.10	2,580.00
Total			15,180.00

Comparative estimates of cost of the 200-foot span

1. WITH CONCRETE FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Item	Quantity	Unit price	Total
Concrete	98.0 cubic yards	$22.00	$2,156.00
Structural steel	250,500 pounds	.06	15,030.00
Reinforcing steel	14,100 pounds	.055	775.50
Total			17,961.50

2. WITH CREOSOTED TIMBER FLOOR AND A DEAD-LOAD ALLOWANCE OF 25 POUNDS PER SQUARE FOOT FOR SURFACING

Item	Quantity	Unit price	Total
Structural steel	246,300 pounds	$0.06	$14,778.00
Creosoted timber	28,500 feet b. m	.10	2,850.00
Total			17,628.00

Case 4.—A bridge which is about one year old includes three 100-foot spans with a roadway of 18 feet. It has a creosoted timber floor with four steel-plate traffic treads 24 inches wide by 3/16 inch thick. The actual cost of this structure at the prices bid for the several items was $8,362.49 for each 100-foot span as shown in the tabulation below.

Recently a bridge of the same dimensions, but with a concrete instead of a timber floor, was designed for construction on a forest road. In each case the design was made for a concentrated live load of two 15-ton trucks with 80 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact. Using the price bid for structural steel in the older bridge and reasonable prices for concrete and reinforcing steel it is estimated that the forest road bridge with concrete floor can be built at a price of $7,906.01 for each 100-foot span as shown in detail below.

Comparison of these designs and the estimates below indicates that it probably would have been possible to substitute for the older design with wooden floor economically designed steel spans with concrete floor at a cost materially less than the price paid for the actual timber-floored structure with steel traffic treads.

The actual cost of the wooden-floored bridge per 100-foot span and the estimated cost of the equivalent structure with concrete floor are as follows:

1. ACTUAL COST OF 100-FOOT SPAN WITH 18-FOOT TIMBER FLOOR AND 4 STEEL-PLATE TRAFFIC TREADS

Item	Quantity	Unit price	Total
Structural steel	96,700 pounds	$0.0687	$6,643.29
Creosoted timber	10,400 feet b. m.	.098	1,019.20
Traffic treads	7,000 pounds	.10	700.00
Total			8,362.49

2. ESTIMATED COST OF 100-FOOT SPAN WITH 18-FOOT CONCRETE FLOOR

Item	Quantity	Unit price	Total
Structural steel	92,300 pounds	$0.0687	$6,341.01
Reinforcing steel	6,800 pounds	.05	340.00
Concrete	49 cubic yards	25.00	1,225.00
Total			7,906.01

BITUMINOUS SURFACED CONCRETE COSTS LESS THAN WOOD WITH HEAVY TRAFFIC TREADS

Case 5.—Two designs were recently prepared for the same span and roadway, 135 feet and 20 feet respectively. Each provided for a concentrated live road of two 15-ton trucks with 80 per cent of their weight on the rear axle, plus an allowance of 30 per cent for impact. One was made for a concrete floor, the other for a creosoted timber floor, each to carry 2 inches of bituminous surfacing.

As shown by the following estimates of the cost of the two designs the one with a concrete floor may be expected to cost $165 more than the wooden-floor design, which is $1.22 per lineal foot and only about 1¼ per cent above the cost of the wooden-floored structure.

Comparative estimates of cost of the two designs

1. WITH CONCRETE FLOOR AND 2-INCH BITUMINOUS SURFACE

Item	Quantity	Unit price	Total
Structural steel	148,600 pounds	$0.069	$10,253.40
Reinforcing steel	9,300 pounds	.055	511.50
Concrete	72 cubic yards	25.00	1,800.00
Bituminous surface	292 square yards	2.50	730.00
Total			13,294.90

2. WITH CREOSOTED TIMBER FLOOR AND 2-INCH BITUMINOUS SURFACE

Item	Quantity	Unit price	Total
Structural steel	153,500 pounds	$0.069	$10,591.50
Creosoted timber	16,440 feet B. M.	.11	1,808.40
Bituminous surface	292 square yards	2.50	730.00
Total			13,129.90

With four lines of steel-plate traffic treads, weighing 9,400 pounds, estimated at the low cost of 8 cents a pound the cost of this structure would be $13,151.90, or only $143 less than the structure with a concrete floor and bituminous surfacing. This estimate is based on the use of 24-inch by 3/16-inch treads which is the usual practice, although it is doubtful if plates so thin will give good service for a long time, especially if the corners of the plates are not bolted down with through bolts. If $\frac{5}{16}$-inch plates are assumed, the estimated cost becomes $13,609.50, which is $314.60 higher than the cost of the bridge with concrete floor and bituminous surfacing, a difference of more than 2 per cent of the total cost in favor of the concrete-floored structure. Without the bituminous surfacing the concrete-floored design would cost only $12,564.90 and would then have a favorable margin of $587 compared with the design with timber floor and thin traffic treads, a difference amounting to more than 4 per cent of the total cost.

OTHER ADVANTAGES OF CONCRETE

It would seem that the above comparisons fully support the declaration by Doctor Waddell in favor of concrete floors from the standpoint of economy. From other points of view it would also seem that there might be added to the three reasons he enumerates, as the basis for his preference, several others, as follows:

1. That a well-shaped and watertight concrete floor so formed as to conduct the surface water to outlets away from the steel work gives better protection from the corrosive effects of weather and dirt to the steel under it.
2. That it is maintained with less labor and that its upkeep, therefore, causes less interference with traffic.
3. That the first cost of a concrete-floored bridge designed to carry heavy truck loads is generally but little more than a timber-floored design for the same live load. Even where prices are unfavorable to concrete the cost may only slightly exceed that of a timber-floored design.
4. A concrete floor increases the rigidity of the spans and in this respect acts as an aid to the bracing.

C. S. JARVIS IS AWARDED MEDAL FOR PAPER ON FLOOD FLOW

C. S. Jarvis, associate highway engineer of the Bureau of Public Roads, has been awarded the J. James R. Croes medal for his paper "Flood-Flow Characteristics" by the American Society of Civil Engineers. This medal is considered as next to the highest honor of this character conferred by the society and is awarded for a paper which is judged worthy of special commendation for its merit as a contribution to engineering science.

Mr. Jarvis has for several years made a special study of flood flow in streams in addition to his regular highway engineering duties. His object has been to arrive at some method whereby the maximum flood flow of a stream can be determined with a fair degree of accuracy from such information as can be collected in the course of a survey for a bridge or river control work.

THE STRENGTH OF MORTAR AND CONCRETE AS INFLUENCED BY THE GRADING OF THE SAND

By T. C. POWERS, Chemist, Department of Materials, Oregon State Highway Commission

> In this article the author discusses the paper published under the same title in the July issue of PUBLIC ROADS from the point of view of the fineness modulus theory and offers an interpretation of the data presented in the original article which he believes to be more in accord with the fundamentals of concrete designing.
>
> He then presents, as a sequel to the original paper and his discussion, a statement of his belief as to what is really wrong with the 1: 3 mortar test.

IN AN article under the above title appearing in the July number of PUBLIC ROADS the following four conclusions are reached:

"1. That there is an ideal grading of sands which will produce maximum strength in concrete.

"2. That the ideal grading curve assumes an arched form showing a predominance of the material retained upon the coarser sieves.

"3. That for a given mix, there is a practical limit to the quantity of material passing each size sieve, where a given strength of concrete is required.

"4. That an exceptionally high tensile strength of sand in 1: 3 mortar is not necessarily associated with a high compression strength of the same material when mixed with the average coarse aggregate in concrete; hence the tensile strength is not a proper gauge of the quality of a sand for concrete."

It was found that for a 1: 2: 4 mix the grading giving maximum strength of concrete in compression was not the same as the grading giving maximum strength in tension in a 1: 3 mortar, and from this fact the conclusion is quite naturally drawn that tensile strength is not a proper gauge of the quality of a sand. However, to the writer, the most important thing illustrated by this investigation is the absurdity of proportioning concrete by an arbitrary mix whenever a given strength is sought. To show this the writer wishes to offer another interpretation of the sand analysis chart published with the article.

Let us consider first the middle ("ideal") curve. While the article does not so state, it is assumed that, since the object was to investigate sand, the coarse aggregate was the same or very similar in all tests.[1] Let us assume, then, that the coarse aggregate has a fineness modulus of 7.5 and a maximum size of 2 inches. (At least 15 per cent retained on the 1½-inch sieve.) From the curve, the ideal sand shows a fineness modulus of about 3.30. Then in a 1: 2: 4 mix the combined aggregate would have a fineness modulus of 6.16, which is about the maximum permissible fineness modulus for the above proportion of cement.[2] To increase the fineness modulus beyond this value would result in an aggregate too coarse for the amount of cement used, and a decrease in strength would result. Furthermore, to decrease the fineness modulus would cause an increase in mixing water for a given consistency with a corresponding decrease in strength.

With this in mind let us examine the top curve of the chart, which is reproduced herewith as Figure 1. The fineness modulus is about 4.00; just about the maximum permissible value for a 1:3 mix with an aggregate graded from zero to four. But, in a 1: 2: 4 mix this sand combined with the same coarse aggregate as before would result in a fineness modulus of 6.40 for the combined aggregate. This is too coarse for the amount of cement used and the strength of the concrete is lower. To equal the strength of the concrete developed by the "ideal" curve the mix would have to be made about one part cement to four parts combined aggregate.

The fineness modulus of the bottom curve is about 2.80. This, combined with the coarse aggregate in a 1: 2: 4 mix would give the combined aggregate fineness modulus of 5.92, which would require more mixing water than the 6.16 of the middle curve. A lower strength is the result.

From the foregoing it should be clear that there is no significance in the fact that the tensile strength of the 1:3 mortar did not reach a maximum as the coarseness of the sand increased, while the compressive strength of the 1:2:4 concrete did. The reason is that the sand did not become too coarse for the 1:3 mortar, but the combined aggregate did become too coarse for a 1:2:4 mix. See Figures 2 and 3.

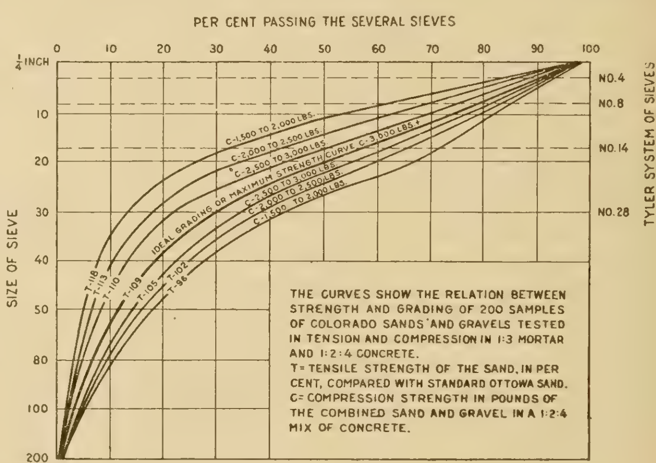

FIG. 1.—SAND ANALYSIS CHART

ANOTHER VIEW OF WHAT THE GRAPH SHOWS

It seems, therefore, that the conclusion that ". . . tensile strength is not a proper gauge of the quality of a sand for concrete" is not justified by the data presented. If the finer or coarser sand were properly combined, the same strength as the "ideal" could be obtained without increasing the cement, all other factors such as structure and cleanliness being equal. Let us illustrate by again taking the top and bottom curves for consideration.

By changing the mix to 1: 2.4: 3.6, using the sand represented by the top curve, the combined aggregate

[1] This assumption is not correct. The coarse aggregate was not the same in all tests. Mr. Rose's study was based upon the results of laboratory tests by the State of Colorado, and it is known that the coarse aggregates represented were not uniform.

[2] See Bul. No. 1, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

would have a fineness modulus of 6.16, the same as the modulus obtained by using the "ideal" sand. All else being equal, the strength would be the same as the ideal of the chart.

By changing the mix to 1:8.8:4.2, using the sand represented by the bottom curve, the combined aggregate would have a fineness modulus of 6.16 and again, all else being equal, the strength would be the same as the "ideal." It is to be noted that in neither case was the proportion of cement increased.

It is quite true that for a 1:2:4 mix, with a given coarse aggregate, a given strength can result from the use of but one and only one grading (fineness modulus) of sand, but it would be only by the remotest chance, if ever, that the sand would happen to be the one showing the maximum tensile (or compressive) strength in a 1:3 mortar.

From this viewpoint the conclusion as to what the graph shows would have to be revised somewhat as follows:

1. That there is no ideal grading of sands which produces maximum strength in concrete, generally speaking, for, with each variation in coarse aggregate, there should be a corresponding change in the fine aggregate, either in its grading (fineness modulus) or in the proportion used.

2. That an exceptionally high tensile strength due to grading only will give the maximum strength only when properly combined with a given coarse aggregate. When used in an arbitrary mix it is just as apt to lower the strength as to raise it.

It is easily seen that to get a certain strength from a given coarse aggregate using an arbitrary mix one's choice of fine aggregate is extremely limited. One

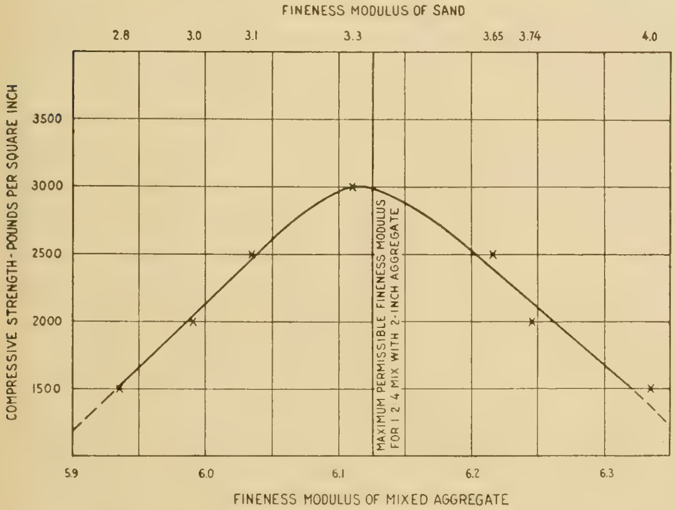

Fig. 2.—Relation Between Compressive Strength of Concrete and the Fineness Moduli of the Sands Represented by the Curves of Figure 1, and the Corresponding Fineness Moduli of the Mixed Aggregate Using Coarse Aggregate With a Fineness Modulus of 7.5 and a Maximum Size of 2 Inches

must either find exactly the right sand or miss the strength desired. How much more sensible and economical it is to design a mix out of the materials available, using the minimum amount of cement with the maximum value of fineness modulus for the conditions at hand. Instead of being confined to a single grading for a given strength, the variety of usable gradings is very large. The proportion between the fine and coarse aggregates should depend entirely upon the nature (grading) of the aggregates and the grading of the combined aggregate desired.

WHAT IS REALLY WRONG WITH THE 1:3 MORTAR TEST

The following argument is based largely upon data contained in Bulletin No. 1 of the Structural Materials Research Laboratory, Lewis Institute; and a bulletin covering nearly the same things issued by the Portland Cement Association. We believe that the water-cement ratio is the fundamental fact underlying all

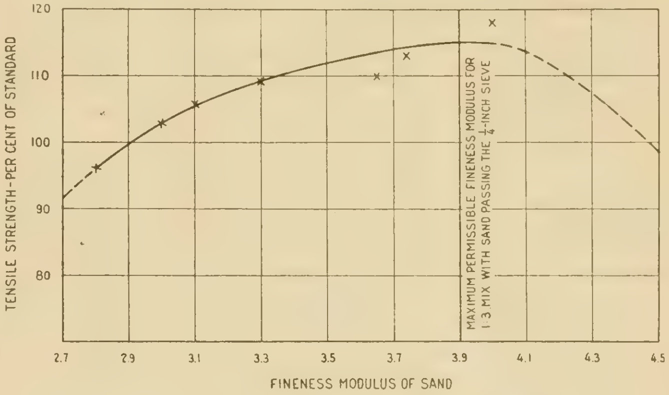

Fig. 3.—Relation Between Tensile Strength of 1:3 Mortar Specimen in Percentage of the Strength of Ottawa Sand Specimens and the Fineness Modulus of the Sands Represented by the Curves of Figure 1

scientific methods of designing concrete mixes, and that the water-cement ratio is a function of the fineness modulus of the aggregate. However, if this theory were only partially correct there would still be considerable weight to the following consideration.

Fine aggregate, or sand, is defined as that material passing the 4-mesh sieve. Our present specifications state that a 1:3 mortar made from a given sand shall develop at least 100 per cent of the strength of an identical mortar made with standard Ottawa sand. In other words, an aggregate having the maximum size of 4 must develop as much strength as Ottawa sand having a fineness modulus of about 2.95 and maximum size of 14. (See Bulletin No. 1 referred to above for a definition of maximum size of aggregate.)

The object of the 1:3 mortar test is to detect structural unsoundness. Occasionally it is said that it determines the suitability of the grading also, but, as will be shown later, a sand suitably graded for one arbitrary mix may be wholly unsuited for some other mix; and, when using the sand in a scientifically designed mix, a sand that will not pass in the 1:3 mortar test may be very well used if combined in the correct proportion—if structurally the sand is sound enough.

Let us see whether with our present method of testing we are able to detect structural unsoundness, or even to determine the suitability of a sand. Examination of hundreds of reports of sand tests shows that the average sand graded from zero to four must have a fineness modulus of about 3 as a minimum in order to develop as much strength as standard mortar. There are finer sands than this that will pass the test, but, as a rule, these sands do not fall into the class of zero-to-four grading. A sand having the same maximum size as Ottawa (14) and having a fineness modulus of 2.5 should, if structurally as sound as Ottawa excel the standard strength. This is due to the fact that 2.5 is the maximum permissible fineness modulus for a zero-to-fourteen aggregate and a 1:3 mix. Ottawa sand has a fineness modulus of about 2.95 and is therefore too coarse to develop its maximum strength. The sand with the 2.5 fineness modulus and maximum size of 14 would therefore excel it. (See table of maximum permissible values of fineness modulus in Bulletin No. 1.) However, the average sand graded from zero to four must have a fineness modulus of about 3 as a minimum in order to pass the test. A modulus smaller than this will raise the water-cement ratio to a point which will disqualify nearly all sands.

It is a significant fact that with a 1:3 mix and an aggregate graded from zero to four the maximum permissible value of fineness modulus is 3.90. This means that sands may range in grading from 3 to 3.90 without chance of failure due to grading. It is, of course, obvious that any sand with a modulus between 3 and 3.90 which fails to pass the test is structurally unsound. But what of the sand coarser than 3 that passes, and the sand finer than 3 that fails? An example or two will help to answer this.

Suppose that we have a sand with a fineness modulus of 3.60, graded from zero to four, which passes the test at 120 per cent. Is the sand structurally sound? We have always assumed that it is because it has developed more than standard strength, but how do we know what it would have done had the fineness modulus been reduced to 3? In other words, the grading of this sand permitted a lowering of the water-cement ratio enough, perhaps, to more than account for the 20 per cent over the standard strength. Suppose, for the sake of argument, that this sand were actually unsound, but that due to the lowered water-cement ratio it was able to pass the test. This is not at all improbable because the difference in the amount of water required for a fineness modulus of 3 and one of 3.60 is considerable. We now, having accepted the sand for use, combine it with a coarse aggregate in such proportions as to produce a certain fineness modulus in the combined aggregate (Abram's method), or we combine it according to some arbitrary mix. Let us consider the first alternative.

EXAMPLES SHOW DEFECTS OF MORTAR TEST

By Abram's method the sand is combined entirely on a basis of grading (fineness modulus), taking into consideration the fineness modulus of the fine aggregate, of the coarse aggregate, and of the combined aggregate to be produced. The aggregates are assumed to be sound structurally. Now that the combination is made we are no longer dealing with a sand graded from zero to four and having a fineness modulus of 3.60, but with an aggregate which has, say, a fineness modulus of 5.8 and a maximum size of 1½. The sand is now an integral part of the combined aggregate and has lost the advantage in grading that it had in the 1:3 mortar test. An inferior concrete is the result because the sand was unsound.

If we adopt the second alternative, and mix the sand in combination with a coarse aggregate according to some arbitrary mix such as 1:2:4 it is possible that not only will the actual unsoundness of the sand lower the strength, but also that *the very grading that caused the sand to pass the test may be a disadvantage in the arbitrary mix.* An example will make this clear. Given:

Coarse aggregate—	
Fineness modulus	7.50
Maximum size	1½
Fine aggregate—	
Fineness modulus	3.60
Maximum size	4
Mix	1:2:4

The above materials combined in a 1:2:4 mix produce a combined aggregate having a fineness modulus of 6.12. Now, the maximum permissible value of fineness modulus for this mix and maximum size is about 5.8. The above mix is therefore too coarse to produce maximum strength. The aggregate is undersanded. There should be either more of this sand added (change the mix to 1:2.6:3.4) or else the same proportion of a finer sand should be used. If the modulus of the sand were 2.4 instead of 3.6 the fineness modulus of 5.8 would be produced without changing the mix. All of this is based on the fact that maximum strength for a given mix is produced when the aggregate has the maximum permissible fineness modulus for that mix. If the value is lowered the water-cement ratio is raised; if the value is raised without increasing the maximum size of the aggregate there is not enough cement to fill the voids.

In the preceding paragraph we found that a sand having a fineness modulus of 2.4 would have produced a maximum strength with that mix and that coarse aggregate. Suppose that a sand graded from zero to four and having a fineness modulus of 2.4 were submitted for test. Let this be a clean, sound, sand. Experience has shown that in all probability this sand would fail in the standard test because of its fineness and consequent raising of the water-cement ratio. The sand would be rejected and in so doing we would be rejecting the only sand that could be combined with the coarse aggregate postulated above to produce the maximum strength for a 1:2:4 mix. This is an important consideration in view of the prevalence of arbitrary mixes. Furthermore, in rejecting this sand, we would be discarding material that might very well be used with any coarse aggregate in a properly designed mixed.

The fallacy in trying to determine the suitability of a sand by the present method is apparent. We have seen how an unsound sand could be accepted for use, and how a sound, useful, sand could be rejected. The gradings suitable in the test might be wholly unsuitable for the concrete, and vice versa. Surely there is a crying need for a test for sand in which the *quality* of the sand is the only variable, and by which the *degree* of soundness can be measured.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORT

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.

DEPARTMENT BULLETINS

No. 105D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1913.
*136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*537D. The Results of Physical Tests of Road-Building Rock in 1916, Including all Compression Tests. 5c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917. 5c.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
*1077D. Portland Cement Concrete Roads. 15c.
*1132D. The Results of Physical Tests of Road-Building Rock from 1916 to 1921, Inclusive. 10c.
1259D. Standard Specifications for Steel Highway Bridges; adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road work.
1279D. Rural Highway Mileage, Income and Expenditures, 1921 and 1922.

DEPARTMENT CIRCULARS

No. 94C. TNT as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

MISCELLANEOUS CIRCULARS

No. 60M. Federal Legislation Providing for Federal Aid in Highway Construction.
*62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-aid Highway Projects. 5c.

FARMERS' BULLETINS

No. *338F. Macadam Roads. 5c.
*505F. Benefits of Improved Roads. 5c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *727Y. Design of Public Roads. 5c.
*739Y. Federal Aid to Highways, 1917. 5c.
*849Y. Roads. 5c.
914Y. Highways and Highway Transportation.

OFFICE OF PUBLIC ROADS BULLETIN

No. *45. Data for Use in Designing Culverts and Short-Span Bridges. (1913.) 15c.

OFFICE OF THE SECRETARY CIRCULARS

No. 49. Motor Vehicle Registrations and Revenues, 1914.
59. Automobile Registrations, Licenses, and Revenues in the United States, 1915.
63. State Highway Mileage and Expenditures to January 1, 1916.
*72. Width of Wagon Tires Recommended for Loads of Varying Magnitude on Earth and Gravel Roads. 5c.
73. Automobile Registrations, Licenses, and Revenues in the United States, 1916.
161. Rules and Regulations of the Secretary of Agriculture for Carrying Out the Federal Highway Act and Amendments Thereto.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 20, D- 4. Apparatus for Measuring the Wear of Concrete Roads.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 10, No. 5, D-12. Influence of Grading on the Value of Fine Aggregate Used in Portland Cement Concrete Road Construction.
Vol. 10, No. 7, D-13. Toughness of Bituminous Aggregates.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL AID HIGHWAY CONSTRUCTION

AS OF

SEPTEMBER 30, 1926

STATES	FISCAL YEARS 1917-1926			FISCAL YEAR 1927									BALANCE OF FEDERAL AID FUND AVAILABLE FOR NEW PROJECTS	STATES
	PROJECTS COMPLETED PRIOR TO JULY 1, 1926			PROJECTS COMPLETED SINCE JUNE 30, 1926			*PROJECTS UNDER CONSTRUCTION			PROJECTS APPROVED FOR CONSTRUCTION				
	TOTAL COST	FEDERAL AID	MILES	TOTAL COST	FEDERAL AID	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES	ESTIMATED COST	FEDERAL AID ALLOTTED	MILES		
Alabama	$ 18,226,411.34	$ 8,725,985.09	1,298.3	$ 1,033,288.73	$ 492,916.35	75.7	$ 4,136,485.77	$ 1,908,310.79	133.1	$ 1,704,203.72	$ 852,101.34	138.8	$ 2,370,141.43	Alabama
Arizona	10,949,878.25	5,863,772.35	729.8	272,263.37	172,004.74	26.7	1,447,874.27	994,768.21	81.0				2,586,703.70	Arizona
Arkansas	18,364,544.50	7,666,698.35	1,323.0	527,162.26	258,399.12	38.8	4,338,696.69	2,052,855.51	279.9	1,250,921.81	621,849.50	94.2	1,006,001.52	Arkansas
California	27,142,596.90	13,003,592.30	1,058.0	2,030,143.39	1,016,180.72	81.6	10,385,227.30	5,079,749.26	248.7	654,219.57	334,197.44	16.0	2,639,095.28	California
Colorado	13,905,904.64	7,127,288.18	745.0	703,326.39	382,146.82	25.5	5,366,219.12	2,579,873.46	235.0	328,001.35	183,954.34	27.9	2,052,549.20	Colorado
Connecticut	5,414,567.19	2,100,585.80	117.1	366,224.19	137,002.12	6.8	4,136,290.50	1,127,030.91	53.5	881,694.24	225,393.21	10.7	743,668.96	Connecticut
Delaware	4,918,052.29	1,781,665.60	124.3	672,626.55	275,874.75	17.5	769,795.50	334,016.25	17.9	243,288.10	71,705.55	8.3	10,795.85	Delaware
Florida	3,832,680.26	1,824,362.32	132.9	982,922.96	489,063.29	31.9	8,943,871.42	4,154,235.83	247.8	658,042.07	331,803.95	10.1	1,285,488.61	Florida
Georgia	24,791,206.97	11,664,237.86	1,794.0	1,551,142.10	761,763.40	91.3	12,161,951.91	5,914,844.55	572.5	100,093.62	50,046.78	12.8	41,060.41	Georgia
Idaho	11,061,198.14	5,882,112.70	724.7	329,995.93	170,379.07	13.3	2,940,022.84	1,774,123.93	166.3	899,794.45	523,839.66	67.4	209,171.64	Idaho
Illinois	44,116,611.86	20,619,995.74	1,377.7	2,012,307.84	980,960.66	68.1	7,992,570.40	3,869,179.54	283.7	1,325,334.89	634,979.47	46.2	3,727,082.59	Illinois
Indiana	16,949,425.87	8,172,125.19	534.3	2,165,077.11	1,022,048.82	65.2	17,281,911.36	8,140,398.89	501.4	105,513.96	52,756.98	0.4	817,025.12	Indiana
Iowa	29,062,375.40	11,926,302.10	2,114.8	1,478,825.10	677,219.97	85.6	12,240,818.19	5,631,714.11	713.9	2,320,724.77	969,866.23	78.3	280,460.59	Iowa
Kansas	32,826,601.64	12,590,489.25	1,160.6	522,940.32	235,898.35	48.9	12,903,599.34	5,060,211.47	703.2	2,777,042.82	969,942.08	134.9	607,869.85	Kansas
Kentucky	20,737,706.10	8,492,082.25	758.3	307,209.39	106,386.83	6.3	7,814,740.29	3,658,046.70	372.7	435,195.46	217,597.72	21.2	738,695.50	Kentucky
Louisiana	13,830,592.68	6,144,739.99	1,055.9	100,810.58	47,898.26	10.0	4,139,744.21	2,009,758.27	197.3	1,212,069.75	517,750.40	38.8	552,261.08	Louisiana
Maine	8,747,552.76	4,192,507.39	303.6	294,012.93	116,469.49	11.6	2,615,984.17	951,520.53	73.3	1,313,319.36	476,921.22	38.1	727,409.37	Maine
Maryland	10,924,943.10	5,112,991.22	423.3				1,350,356.90	641,523.57	72.2	426,154.66	170,540.72	16.1	1.49	Maryland
Massachusetts	18,353,757.71	6,657,260.62	374.5	58,723.21	22,710.05	0.2	5,208,753.35	1,397,642.65	73.3	773,451.02	198,908.17	11.9	1,832,204.51	Massachusetts
Michigan	25,997,240.78	11,827,052.30	963.0	390,946.77	188,717.04	13.6	13,317,248.82	6,086,174.43	378.7	423,878.96	151,472.69	4.7	2,088,948.54	Michigan
Minnesota	37,170,985.95	15,586,116.56	3,181.9	2,469,492.66	1,130,159.07	228.6	7,383,336.64	2,724,700.00	362.6	407,723.04	6,000.00	16.2	144,804.37	Minnesota
Mississippi	15,146,088.52	7,414,534.10	1,129.0	769,794.60	377,270.65	45.3	7,619,272.14	3,717,199.28	381.9	581,391.39	283,814.10	48.4	335,199.87	Mississippi
Missouri	28,989,166.92	13,736,014.85	1,543.2	4,214,395.17	1,841,217.99	123.8	16,904,839.84	6,712,627.78	462.0	671,100.91	291,025.70	13.4	205,549.68	Missouri
Montana	11,400,983.81	6,333,465.89	1,054.9	384,690.63	258,568.03	48.9	2,405,431.31	1,900,630.92	166.5	861,904.17	476,823.50	102.0	4,455,396.66	Montana
Nebraska	11,533,401.62	5,474,202.52	1,768.3	593,394.96	291,575.09	65.7	13,823,916.11	6,814,242.74	1,448.9	518,097.23	258,819.11	69.2	1,796,395.54	Nebraska
Nevada	7,558,195.51	5,130,934.59	538.8	1,219,911.65	1,062,768.79	115.0	2,470,999.94	2,119,521.70	290.2	36,304.46	31,846.26	4.9	450,143.66	Nevada
New Hampshire	4,992,558.60	2,377,450.07	237.6				1,401,209.12	640,606.43	41.8	81,083.86	35,258.94	1.1	116,176.56	New Hampshire
New Jersey	16,346,301.01	5,098,342.21	290.3	863,631.14	202,054.20	13.3	8,170,401.15	2,721,490.84	44.1	1,291,908.66	298,965.00	19.9	146,567.75	New Jersey
New Mexico	12,404,337.77	7,339,657.38	1,427.0				2,371,922.68	1,588,165.05	161.2	501,479.13	340,045.71	63.5	1,704,517.86	New Mexico
New York	43,224,279.79	17,911,957.19	1,197.0	1,875,003.91	676,396.18	42.0	35,703,573.00	9,594,450.20	607.1	9,236,650.00	2,087,355.00	129.1	3,775,036.43	New York
North Carolina	27,009,419.47	11,177,337.94	1,257.9	3,621,683.44	1,412,891.77	73.3	4,970,040.28	2,262,344.60	132.7	972,106.70	456,189.10	40.4	408,442.59	North Carolina
North Dakota	12,313,311.40	6,031,859.78	2,193.1	1,012,765.16	469,539.29	102.6	6,880,714.72	3,641,076.33	932.6	1,058,913.12	538,327.89	148.9	67,855.71	North Dakota
Ohio	47,699,532.90	17,371,787.03	1,364.1	1,298,371.21	554,351.87	49.5	12,506,409.27	4,841,077.91	370.3	2,745,719.70	930,242.79	71.6	2,034,336.40	Ohio
Oklahoma	28,247,950.33	13,159,999.15	1,178.9	679,879.55	331,291.15	27.7	2,422,693.50	1,128,971.02	99.7	1,873,582.72	845,342.12	131.9	594,183.56	Oklahoma
Oregon	17,027,878.42	8,593,214.79	939.2	702,784.65	403,640.92	23.7	3,018,523.75	1,635,338.95	120.6	232,363.19	139,417.91	7.4	107,734.43	Oregon
Pennsylvania	61,366,150.80	21,560,732.04	1,188.8				28,932,350.11	8,216,400.41	578.7	1,476,142.34	454,809.16	28.0	1,106,839.39	Pennsylvania
Rhode Island	3,988,616.09	1,558,829.06	86.7	207,117.54	86,175.00	5.7	1,282,981.70	340,980.00	22.8	743,891.14	201,195.00	13.4	480,389.94	Rhode Island
South Carolina	15,020,639.90	6,766,322.93	1,481.9	731,575.09	262,587.41	44.2	5,923,078.58	2,673,278.57	205.1	309,121.16	55,921.58	13.4	43,413.51	South Carolina
South Dakota	17,468,373.19	8,603,826.97	2,161.2	577,690.35	289,877.22	127.1	4,026,484.17	2,041,920.59	676.2	79,488.88	39,744.44	14.1	191,420.78	South Dakota
Tennessee	21,624,631.57	10,276,584.02	780.0	757,678.17	353,743.87	21.3	8,490,357.75	3,770,336.51	243.6	390,741.12	185,871.09	8.0	694,055.51	Tennessee
Texas	69,183,673.48	27,440,254.72	4,920.2	2,092,131.01	939,622.50	165.2	17,143,916.84	7,553,979.60	764.6	3,203,769.21	1,494,511.91	87.4	3,178,062.27	Texas
Utah	8,253,178.03	5,098,440.68	546.4	72,006.59	53,284.87	9.1	1,883,615.88	1,417,258.71	170.4	853,136.36	656,272.37	49.7	593,522.37	Utah
Vermont	4,242,042.64	2,017,699.51	134.5				2,120,394.45	819,045.58	39.2	64,871.97	24,914.65	0.8	406,847.26	Vermont
Virginia	21,990,249.44	10,385,728.11	1,005.5	761,010.78	359,368.50	28.9	6,146,256.68	2,613,398.01	172.4	67,455.62	26,640.23	0.9	116,379.15	Virginia
Washington	17,078,511.63	7,782,909.46	668.6	267,789.25	101,642.49	4.3	3,303,868.75	1,635,600.00	43.5	1,008,163.90	374,000.00	43.7	251,624.05	Washington
West Virginia	9,473,716.44	4,141,062.65	392.9	457,126.16	193,696.41	12.5	5,569,201.30	2,196,923.30	153.8	985,481.20	454,756.64	45.0	366,072.00	West Virginia
Wisconsin	24,856,508.19	10,392,705.73	1,592.1	101,729.42	50,664.71	9.3	8,467,314.02	4,034,936.12	379.1	591,446.04	270,722.50	24.4	2,689,785.94	Wisconsin
Wyoming	10,928,302.56	6,040,887.05	1,133.5	465,830.26	297,267.00	60.8	2,739,880.13	1,741,877.34	206.2	78,837.13	50,612.00	19.1	435,630.61	Wyoming
Hawaii							1,050,897.93	312,635.18	15.9				787,517.82	Hawaii
TOTALS	966,692,834.36	426,178,703.58	52,526.6	41,987,432.47	19,553,694.83	2,166.4	364,636,044.09	154,776,992.53	14,699.0	48,755,818.93	18,865,072.15	1,992.6	52,000,536.91	TOTALS

* Includes projects reported completed (final vouchers not yet paid) totaling: Estimated cost $ 90,739,378.86 Federal aid $ 38,751,808.47 Miles 3,390.4